P. 530
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PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
SEPTEMBER 1954

ORDINARY MEETING

16 March, 1954

WILFRID PHILIP SHEPHERD-BARRON, M.C., T.D.,
President, in the Chair

The Council reported that they had recently transferred to the class of

Members

Austin, William Herbert, B.Sc.(Eng). (*London*).
Bard, Laurence Percival, B.Sc.(Eng.) (*London*).
Benham, Alan Dudley, *M.B.E.*, M.Sc., B.E. (*New Zealand*).
Black, Edwin, B.Sc.(Eng.) (*London*).
Collins, Arthur Richard, *M.B.E.*, D.Sc.(Eng.) (*London*), Ph.D. (*London*).
Peat, George, B.Sc. (*Edinburgh*).
Priestley, William Leslie, B.Sc.(Eng.) (*London*).
Ratcliff, William Watts.
Shaw, Emil Archibald, B.Sc.(Eng.) (*London*).

and had admitted as

Graduates

Aggarwal, Hari Dev, B.Sc.(Eng.) (*London*), I.C.E.
Alcock, Donald Geoffrey, B.A. (*Cantab.*).
Arah, Robert Moore, B.A. (*Cantab.*).
Banjo, Jamieson Ademola, B.Sc.Tech. (*Manchester*).
Blumenthal, Arthur Harold, B.Sc. (*South Africa*).
Boldero, John Wearmouth, Stud.I.C.E.
Bower, Frank William, B.Eng. (*Liverpool*).
Bowyer, Colin, Stud.I.C.E.
Boyd, Robert Reginald, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
Braithwaite, Arthur Stuart, Stud. I.C.E.
Brett, George Edmund, Stud.I.C.E.
Brownlee, Gilbert Hall, B.Sc. (*Glasgow*), Stud.I.C.E.
Bruce, Frederick Robert, B.Sc. (*Leeds*).
Butcher, Donald Bernard, B.Sc. (*Bristol*), Stud.I.C.E.
Campbell, Charles Niall Kennedy, B.Sc. (*Cape Town*).

Campbell, Donald Ewen McVicar.
Carpenter, David Gillard, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Chalmers, David John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Chapman, Colin Leonard, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Close, Kevin David Patrick, Stud. I.C.E.
Cohen, Marshall, B.A., B.A.I. (*Dublin*).
Cooper, Michael John, B.A. (*Cantab.*).
Cope, Bryan Ivan, B.A., B.A.I. (*Dublin*).
Cornfoot, William Bright, B.Sc. (*Durham*).
Crowter, Philip Edward Wallace, B.Sc.(Eng.) (*London*).
Davies, David Charles, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Donald, Ian Mudie, B.Sc. (*St Andrews*), Stud.I.C.E.
Doody, Michael Charles, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Dunn, Geoffrey Gladstone, B.E. (*New Zealand*).
Elliott, Douglas Roberts, B.Eng. (*Liverpool*), Stud.I.C.E.
Evans, David Egerton, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Fair, Eric James, B.Sc. (*Glasgow*), Stud.I.C.E.
Falconer, Ian McLeod, B.Sc. (*Aberdeen*).
Farquhar, Harry Stiven, B.Sc. (*Glasgow*).
Ferguson, David, Stud.I.C.E.
Fletcher, Tom Walter, B.Sc. (*Nottingham*), Stud.I.C.E.
Forsyth, Finlay Macdonald, B.Sc. (*Glasgow*), Stud.I.C.E.
Gardiner, John Muir, B.Sc. (Glasgow).
Gardner, Gavin, Stud.I.C.E.
Gilbert, Graham David, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Halley, Alexander Bell, B.Sc. (*Glasgow*).
Hare, Victor Dennis, B.Sc. (*Wales*), Stud.I.C.E.
Harrop, Jack, B.Sc. (*Leeds*), Stud.I.C.E.
Horne, Michael John Barton, Stud. I.C.E.
Hubbard, Geoffrey Frank.
Hurst, Thomas Anderson, B.A. (*Cantab.*).
Ibbotson, Richard Errol Jeremy.
Jagger, Denis Hugh, Stud.I.C.E.
Jenkins, Francis, B.Sc. (*Wales*), Stud. I.C.E.
Jones, William Farbridge, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Kaczkowski, Tadeusz Adam, Stud. I.C.E.
Kay, John Basil Richard, B.Eng. (*Sheffield*), Stud.I.C.E.
Kirkor, Andrzej, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lambert, John Anthony, B.A. (*Cantab.*).
Leather, Donald, B.Eng. (*Liverpool*), Stud.I.C.E.
Lee, Philip Michael.
Leech, John, B.Sc.Tech. (*Manchester*).
Lees, James, B.Sc. (*Edinburgh*), Stud. I.C.E.
Llewelyn, Frederick John, B.Sc. (*Wales*), Stud.I.C.E.
Luckett, Geoffrey Albert James, Stud.I.C.E.
McCleery, James, B.Sc. (*Belfast*), Stud. I.C.E.
Macdonald, Alastair Kinross, M.A. (*Cantab.*), Stud.I.C.E.
Macdonald, Allan Joseph, B.E. (*New Zealand*).
McKinlay, William, Stud.I.C.E.
Macleod, John David Campbell, B.Sc. (*Cape Town*).
Madawela, Joseph Edward Oliver, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Marshall, Ivor Durose, B.Sc. (*Wales*).
Massias, Manesé, Stud.I.C.E.
May, Patrick Joseph, B.E. (*National*).
Mendes, Denis Francis, B.Sc. (*Glasgow*), Stud.I.C.E.
Menzies, Bruce George Alexander, B.E. (*New Zealand*), Stud.I.C.E.
Minshull, Leonard, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Mort, Thomas Edward, B.E. (*Sydney*).
Murphy, William Edmund, B.Sc.(Eng.) (*London*), Stud.I.C.E.
O'Donovan, Diarmuid Colmcille, B.E. (*National*), Stud.I.C.E.
O'Donovan, Matthew Brendan, B.E. (*National*).
Oldfield, John Benjamin, B.A. (*Cantab.*), Stud.I.C.E.
Owen, Reginald St John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Palmer, John Edward, Stud.I.C.E.
Parsons, Patrick.
Pollard, James Michael, B.A., B.A.I. (*Dublin*).
Poore, David Cyril Rodney, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Randall, Keith George, Stud.I.C.E.
Ray, William Joseph Fergus, B.A. (*Oxon.*), Stud.I.C.E.
Richmond, Robert Leo, B.Sc. (*Aberdeen*), Stud.I.C.E.
Robinson, George.
Robinson, Peter, B.Eng. (*Sheffield*), Stud.I.C.E.

SAMARASURIYA, GAMINI DHARMAKIRTI, B.Sc.(Eng.) (*London*).
SCOTT, WILLIAM PIRIE, B.Sc. (*Glasgow*), Stud.I.C.E.
SCUTT, ALAN JOHN, Stud.I.C.E.
SHAND, JOHN, B.Sc. (*Edinburgh*).
SHIELDS, DOUGLAS JOHN, B.Sc. (*Bristol*).
SPARROW, VICTOR NORMAN, B.Sc.(Eng.) (*London*), Stud.I.C.E.
THOMPSON, JAMES LAURENCE, B.Sc. (Eng.) (*London*), Stud.I.C.E.
TORK, ANDRES, B.E. (*New Zealand*).
TURNER, DOUGLAS ARNOLD, B.Sc.(Eng.) (*London*).
TURNER, JAMES KENNETH, B.Sc. (*Wales*).
WEBSTER, KENNETH CAMPBELL, Stud. I.C.E.
WHATLEY, MICHAEL JOHN, B.Sc.(Eng.) (*London*).

and had admitted as

Students

ANDREWS, DAVID JOHN.
ANTROBUS, ARTHUR.
BOUTWOOD, ROBERT MORRIS.
BOYS, JOHN ALFRED.
BUCHANAN, JOHN.
BURGESS, MICHAEL ROBERT ASHBY.
BURY, RONALD MARK.
CARR, JAMES ROBERT MONCRIEFF.
CHAPMAN, MALCOLM HERBERT.
CHARTERS, JAMES.
COOPER, JOHN RATCLIFFE.
CORFIELD, JOHN.
CORNFORTH, DEREK HARLAND.
COTTRELL, JOHN KENDRICK.
DEARDEN, ERIC HOWARD.
DICKMAN, BERNARD JOSEPH.
EDMONDS, DAVID THOMAS.
GEE, ANTHONY FRANCIS.
GRANT, ANTHONY CARR.
GREGORY, JOHN WILLIAM.
GRIFFITHS, DONALD MICHAEL.
HAMILTON, GEORGE THOMSON JACKSON.
HARRADINE, JOHN MICHAEL.
HARRINGTON, JOHN FREDERICK.
HARRIS, IVOR LOVELL.
HAYWORTH, ALEXANDER LIDDELL.
HILL, RICHARD INGLIS.
HOBBY, JOHN GEOFFREY.
HOLT, DAVID ALAN.
HOUSTON, MICHAEL.
HURLE, MICHAEL.
HYDE, JOHN MACLEOD.
INGLE, JOHN DAVID.
JOHNSON, LANE BRYAN.
JONES, BRIAN GEOFFREY.
KILGOUR, DAVID MCNAUGHT.
KINGSWOOD, DENIS.
KNOWLES, JOHN THORNTON.
LEWIS, CONRAD WALFORD.
LOCHHEAD, JOHN ALLAN.
MCDONALD, CHARLES HARDING.
MENIRU, AUGUSTINE EBELE OKAFOR.
MITCHELL, PETER.
MORRIS, MAURICE COLIN.
MORTON, MICHAEL DIARMAID MATTHEW.
MURGATROYD, DEREK ERNEST.
NOWELL, IAN PHILIP.
PARKES, DOUGLAS BRIAN.
PHILLIPS, JOHN HAYDN.
PHILPOTT, KEITH LEONARD.
PITT, ROGER CHARLES EDWARD.
QUIGLEY, LEONARD.
RAWSON, DEREK.
RICHARDSON, BRIAN.
ROBERTS, DAVID LEWIS.
ROBERTS, RONALD JOHN.
ROBINSON, JAMES MICHAEL.
ROWE, DAVID AUGUR.
SAIZ-AMIGO, OSCAR.
SHAW, JAMES MAXWELL.
SIMMONDS, HARRY KEITH.
SIMPSON, JAMES.
SKILLING, JOHN.
SKLODOWSKI, WLADYSLAW JOZEF JAROSLAW.
SMITH, ERNEST WALTER.
SMITH, PAUL JOHN.
STEARS, HAMISH THOMAS.
STONES, FREDERICK BARRY.
STROK, WOJCIECH.
SUMMERS, JOHN EDWIN.
SWORD, JOHN WATT.
THOMPSON, PETER ALLAN.
TONGE, GEOFFREY HAROLD.
TOZER, PETER RICHARD.
VAUGHAN, BARRY EVAN JONES.
WEDLAKE, DAVID HARRY.
WEIR, WILLIAM.
WILLIAMS, PETER JEFFREY.

The following Paper was presented for discussion and, on the notice of of the President, the thanks of the Institution were accorded to the Authors.

Paper No. 5994

"Woodhead New Tunnel: Construction of a Three-Mile Main Double-Line Railway Tunnel"

by

Peter Adamson Scott, B.Sc., M.I.C.E.

and

John Isdale Campbell, M.I.C.E.

SYNOPSIS

Deterioration of the linings of the 100-year-old twin single-line tunnels carrying the railway between Manchester and Sheffield through the Pennines at Woodhead led to the decision to replace them with a new double-line tunnel, more than 3 miles long, lined throughout with a minimum thickness of 21 inches of plain concrete.

An overhead wiring system was incorporated in the design of the new tunnel, and its completion was phased with the electrification of the Manchester–Sheffield–Wath lines.

The contract for the work was let on a target basis, with a fixed fee for the contractor adjusted according to the relation between actual and target costs. Construction started in February 1949. The difficulties encountered during construction justified this departure from the orthodox type of contract.

A novel and attractive method of excavation, by working radially from a 12-foot-by-12-foot pilot tunnel located in the centre of the 30-foot-wide-by-24-foot-high section was tried out, but proved to be impracticable in the bad shale encountered, and undesirable even in the sandstone regions. To achieve the necessary progress in excavation, access to more working points than was afforded naturally by the single shaft and two portals had to be arranged, and for this purpose a system of haulage ways parallel to the main drive and connected to it at intervals was driven. Up to as many as nine working points for the enlargement of the pilot tunnel to full section were operated at one time.

The amount of immediate support needed for the roof in the exceptionally weak shales and sandstone penetrated was difficult to forecast, and two serious roof falls occurred. The work was completed in October 1953, at a cost of £4¼ million.

HISTORICAL

OF the many problems which present themselves to the railway civil engineer, the most insistent is probably that of the obsolescence of structures; the existing twin single-line tunnels at Woodhead are notable examples.

These tunnels are situated on the section of the former Great Central Railway connecting Manchester and Sheffield. The construction of this line by the Sheffield, Ashton-under-Lyne, and Manchester Railway Company was authorized by Act of Parliament in 1837 and the first sod was

cut by the Chairman, Lord Wharncliffe, at Saltersbrook near Woodhead on the 1st October, 1838. Construction work proceeded under the direction of Mr C. G. Vignoles until November 1839 when he resigned, and was followed by Mr Joseph Locke.

The location of the line passed through the Pennine Range in tunnel (see *Fig. 1*) and, owing to financial strictures, only a single-line tunnel was constructed in the first instance with the intention of later driving a second tunnel, if justified by revenue.

Various sections of the railway were opened to traffic until, in July 1845, only the completion of the tunnel remained to link up Manchester and Sheffield. This was accomplished in December, and the line opened to traffic on the 23rd December, 1845. Work on the construction of the second tunnel was commenced in 1847, and completed in February 1852.

Both these tunnels are, therefore, now more than 100 years old and generally speaking have given good service. Inevitably, however, deterioration has taken place, making itself increasingly evident during the past 20 years, principally in the disappearance of the mortar in the joints of the stone lining.

With an intensity of traffic of more than 80 trains per day in each direction, maintenance work on the tunnel lining was difficult and, with only the usual week-end line possessions, could not be undertaken to a sufficient extent to keep pace with the rapidly worsening situation.

In 1946 it was decided that the engineer should have absolute possession of each tunnel alternately for a period of 9 months to carry on repair work continuously by day and night shifts. This involved single-line working through one tunnel, and the diversion of a very considerable volume of traffic over other routes. At the end of the agreed period, it was obvious that to overtake all the essential repair work, an appreciable extension of time would be required. The traffic-operating difficulties were such that this could not be granted, and in view of the age and condition of the tunnels, it was considered that the only practicable alternatives were :—(1) To construct a new single-line tunnel and repair one of the existing tunnels. (2) To construct two new single-line tunnels. (3) To construct a new double-line tunnel. On investigating these 3 alternatives, the estimated costs were found to be in the proportion 1·04 : 1·43 : 1·00 respectively. Taking all circumstances into consideration, the engineers recommended to the management of the London and North Eastern Railway that a double-line tunnel should be constructed. This recommendation was accepted by the Board of Directors, and authority to proceed was given by the newly constituted Railway Executive on the 15th November, 1948. Parliamentary authority for the scheme was contained in the London and North Eastern Act of 1947, which, for the protection of certain bodies and authorities, included a number of clauses which gave rise to additional works and costs.

In the case of Manchester Corporation, for the protection of the

Waterworks Catchment Area and the domestic water supply reservoirs which are on the west side of the tunnel (Clause 13 of the Act), it was required that all water arising from the new tunnel should be purified before being discharged into the river Etherow, which feeds the reservoirs. It was also set down that the Corporation's Medical Officer would have the right to examine any person employed in the construction of the works to avoid risk of contamination of the water supply.

A further stipulation of the Act required the provisions of the Town and Country Planning Acts to be observed ; this particularly related to the siting and formation of spoil tips.

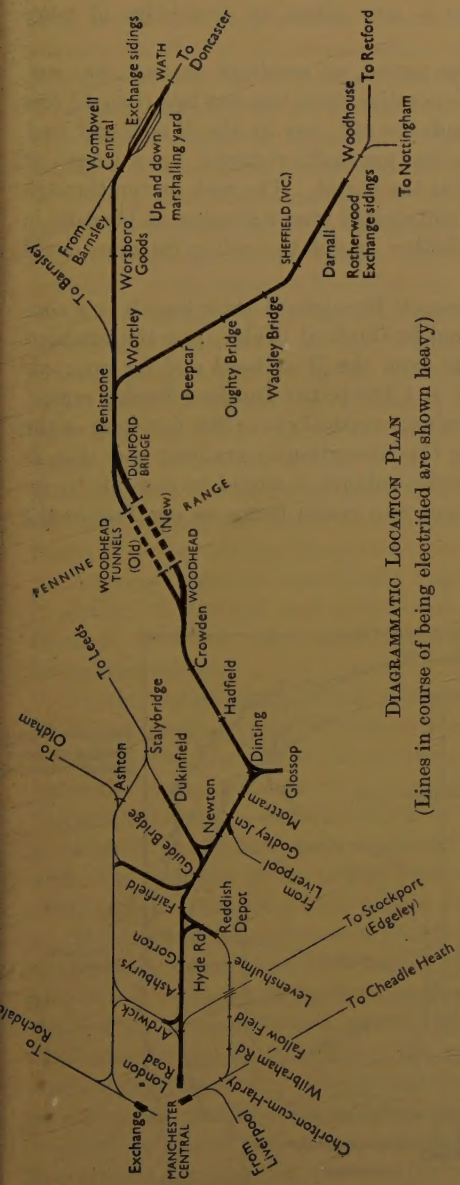
Location and Design

With the existing railway lines to work from, topographical information was easy enough to obtain, although when the engineers carried out the Parliamentary survey in the notoriously severe winter of 1946/7 some normally prominent landmarks had to be located by probing in deep snowdrifts with a ranging rod. Details of the nature and behaviour of the rock to be penetrated were more difficult to obtain, and no technical account of the planning and execution of the old tunnels could be traced. One record, however, had survived ; this was a large and detailed longitudinal section along the original tunnel and through the 5 shafts, made by the Resident Engineer (Mr W. A. Purdon) in 1845 and deposited in the Geological Museum, South Kensington. It described the strata encountered, named the fossils, and indicated the limits and locations of the geological features.

Such a record, relating to ground adjacent to his operation, is an unusual gift for any engineer about to drive a tunnel. Full use was made of the indications given in the document, and valuable deductions were possible, but for reasons given later in the Paper many of the conditions actually encountered could not be predicted from the geological section.

After careful consideration of the geological formation and the condition of the old tunnels, the existing track lay-outs, and possible traffic operating requirements, it was decided to locate the new tunnel on the south side of the existing tunnels at a distance of 100 feet between the centres of the existing down-line tunnel and the new tunnel. The existing tunnels are straight throughout their length of 3 miles 22 yards and the distance of 100 feet is maintained throughout the length of the new tunnel, except for a length of 600 feet through solid blocky sandstone at the Woodhead end where it is on a curve of 40 chains radius to permit the new tracks to be connected to the existing alignment.

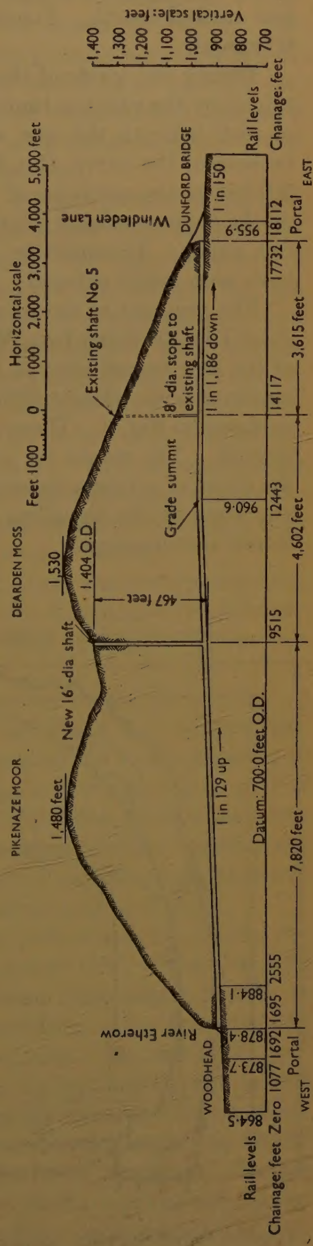
The portals of the new tunnel were located at the points where rock cover became sufficient for tunnelling, and it so happened that, on the line selected, this produced a longer tunnel than the existing ones. The extra length, though only 131 feet (the difference between 15,906 feet and 16,037



Diagrammatic Location Plan

(Lines in course of being electrified are shown heavy)

Fig. 2



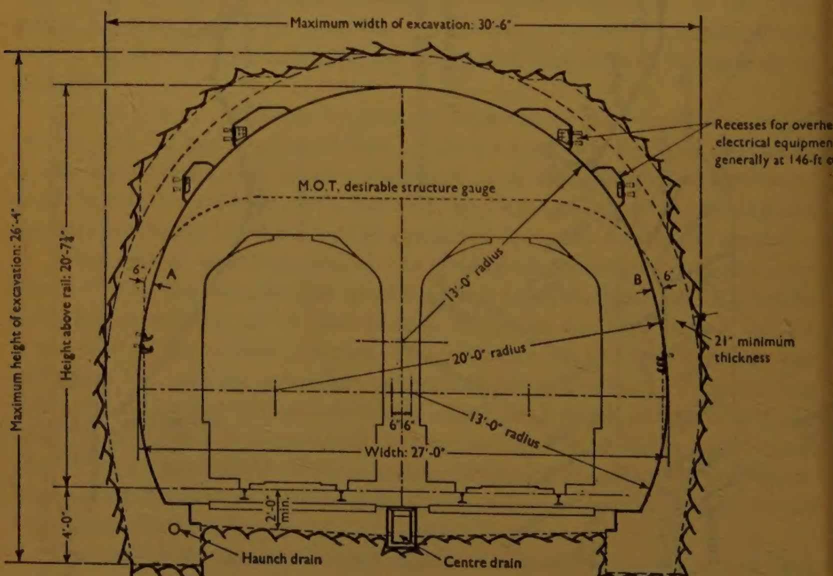
Longitudinal Section

feet) was sufficient to make the new Woodhead tunnel (3 miles 66 yards) the third longest railway tunnel in the country, whereas the existing tunnels had ranked fourth. Fourth place is now taken by Standedge (3 miles 60 yards).

During the whole of the driving period, no resultant damage was sustained by the existing tunnels. Generally, there was not less than 77 feet of rock between the new work and the old, but at the Woodhead end, because of the curve, this distance was gradually reduced, for a length of 600 feet to the minimum of 27 feet at the portal. The rock driven through on the curve was hard sandstone, and special care was taken in blasting in this region. It cannot be said whether or not the ruling distance could have been safely reduced.

The gradient of the existing tunnels throughout their length is a constant 1 in 203 rising from Woodhead to Dunford Bridge, but the gradient of the new tunnel rises at 1 in 129 from the Woodhead end for approximately 2 miles, thence falling at 1 in 1,186 to the Dunford Bridge portal, as shown in *Fig. 2*. There is a parabolic vertical curve 800 feet long at the summit. The reason for adopting this alteration in gradient was that in the event of a future increase in traffic volume, it would be possible by an alteration to the signalling to permit two trains being on one line in the tunnel simultaneously.

Fig. 3



Scale : 1 inch = 10 feet

Typical Cross-Section, showing M.O.T. Structure Gauge and Eastern Region Stock Gauge

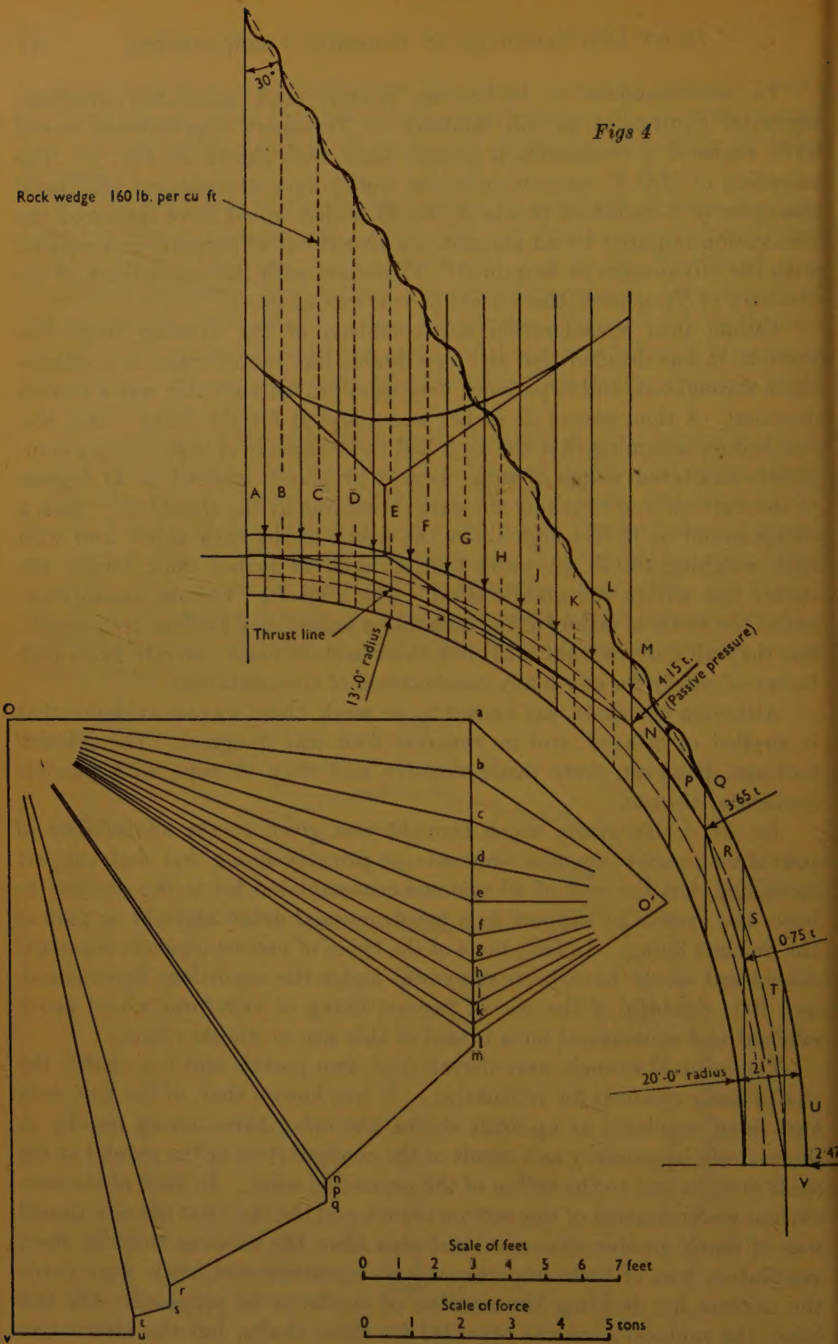
To accommodate a double-line railway and associated overhead electrical equipment to full Ministry of Transport requirements would have required a cross-section greater than that shown in *Fig. 3.* The adoption of M.O.T. recommendations would have necessitated additional clearance of 6 inches at points A and B, which would have increased the excavation required by an amount considered out of proportion compared with the advantages to be gained. Therefore, with the concurrence of the Ministry of Transport, the section shown was adopted.

Taking into consideration the condition of the existing single-line tunnels, it was decided that the new double-line tunnel must be concrete-lined throughout, and experience very soon proved that this was a correct decision. A thickness of 21 inches was adopted for the lining; this was checked by assuming that the arch had to be capable of supporting a completely shattered wedge of rock, formed by planes inclined at 30 degrees to the vertical and touching the excavation line at the shoulders. Such a wedge would be 15 feet high above the crown of the excavation, and with rock weighing 160 lb. per cubic foot, an arch 21 inches thick retains the thrust line within the middle third. (See *Figs 4.*) Various assumptions about the mode of failure of the arch under excess rock loading are possible, but they all indicate that a 21-inch-thick arch of mass concrete has a high factor of safety for any likely combination of circumstances.

Although the shale was known to be weak, there was no evidence that it swelled or heaved, and no concrete floor was designed. The sidewall footings, however, were made massive and deep to cope with possible horizontal thrusts.

In the design stage, much thought was given to the advisability of providing precast concrete segments as primary lining and rock-support combined, but the cost of all systems contemplated led to the decision to leave the system of support as a constructional detail and not as part of the finished lining. In fact, none of the types of precast concrete segments considered would have been successful under the conditions experienced, and it is doubtful if the use of precast lining of any form would prove efficient and economical for a tunnel of this size in similar strata.

The original tunnels were driven from two portals and five shafts, the shafts being retained for ventilation. It was known that, of the five, only two acted regularly as up-draft shafts, the other three acting usually as down-draft, apparently as a result of the configuration of the ground at the shaft mouths and to the action of the prevailing wind. In view of the concurrent electrification of this section of line, and the fact that the new tunnel was of much greater cross-sectional area than the existing tunnels, shaft ventilation was of little importance and constructional needs were made the criteria for deciding the number of shafts to be provided. On this basis, the engineer's scheme provided for three shafts, but the alternative proposals submitted by the contractors allowed for only one construction shaft. The proposals were accepted but it was decided to provide



Thrust Line and Funicular Polygon for Arch Analysis

ventilation near the gradient peak, by driving a 45-degree stope, 8 feet in diameter, to connect with the existing No. 5 shaft. (*Fig. 2*, p. 509.)

The centre of the 16-foot-diameter working shaft was sited 25 feet 6 inches to the south of the centre-line of the tunnel to keep the hoisting arrangements clear of the tunnel operations, and also to provide a ventilation shaft clear of the running lines to avoid the danger to trains of any objects being dropped down the shaft. From a constructional point of view this offsetting of the shaft proved advantageous.

The contractor's proposal to use only one construction shaft from which to drive the pilot tunnel, and thereafter to enlarge to full section from only the two portals made it necessary to re-plan the disposition of the spoil tips. This involved considerable negotiation with the authorities who had obtained protection under the Act. The bodies charged with the duty of preserving the amenities of the countryside were firm in their demands regarding the disposal of the spoil but, with understanding and a spirit of give-and-take on both sides, agreement was reached and the spoil tips today present a very acceptable appearance. No figures are available to assess the cost of achieving this aesthetic near-ideal, but extra cost was involved and the country's ability to pay for such perfection in relation to development is questionable.

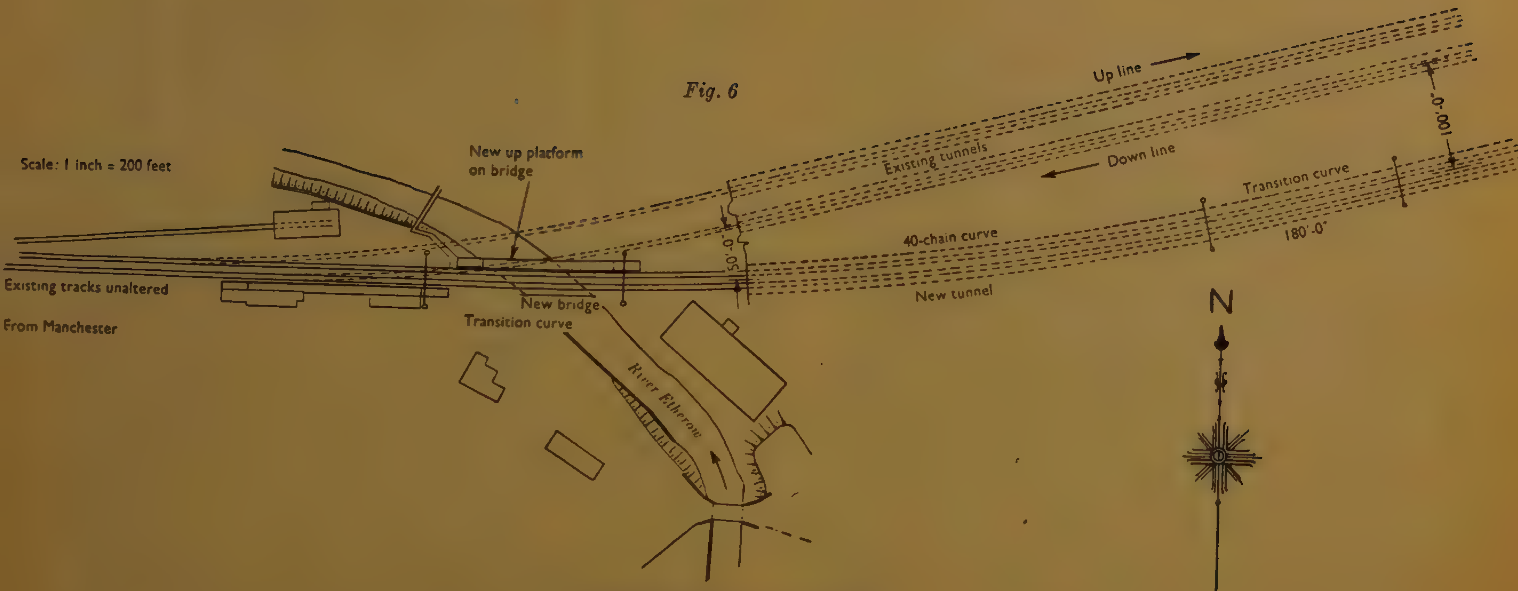
Ancillary Works

The approach works to the new tunnel do not involve any items of outstanding interest. Both Woodhead and Dunford Bridge stations required remodelling and the lay-outs are shown in Figs 5, Plate 1, and *Fig. 6*. Curves of 40 chains radius, with transition curves 180 feet long, were adopted at each end to connect the selected line of the tunnel to the existing tracks.

The divergence of the old and new tracks at the point of crossing the River Etherow was so small that the existing plate-girder bridge carrying the down track fouled a large part of the up side of the new bridge. It had been intended to build only the down line of the new bridge before diverting the down traffic to the new tunnel. The old down-line bridge could then have been removed to make way for the rest of the new bridge, but this would have involved a period during which steam traffic would have had to pass through the new tunnel. It was found that by accepting a 15-chain curve just outside the old portal and a 19-chain radius connexion beyond the bridge in the existing tracks there was just room to slew the old down-line bridge clear of the essential decking of the new bridge (as can be seen from *Fig. 7*) and thus allow both new tracks to be laid over the river ready for the complete change-over of traffic through the new tunnel on the introduction of the electrified services.

The new bridge consists of deck units formed of R.S.J. frames encased in concrete and laid between two piers and abutments in three equal spans,

Fig. 6



APPROACH LAY-OUT AT WOODHEAD

and it carries the new up platform in addition to the two tracks. The lay-out is indicated in *Figs 8.*

In order to comply with the provisions of the Act, a water-treatment plant, indicated in *Figs 9,* was constructed adjacent to the Woodhead portal to ensure that all water draining out of the new tunnel should reach the river Etherow in a purified condition, and without excessive silt.

Fig. 7

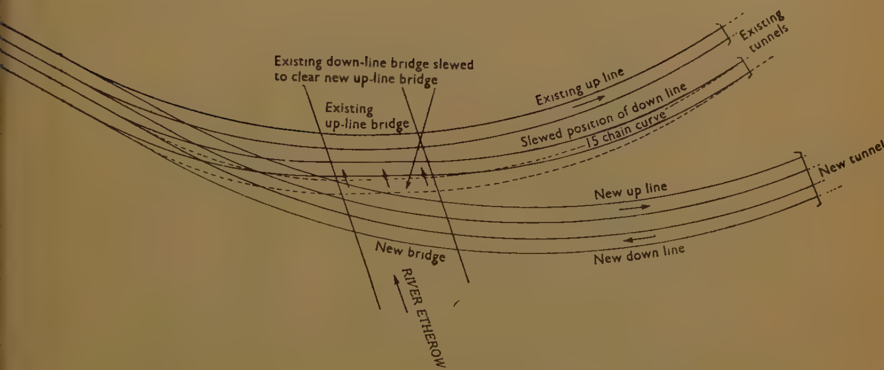


DIAGRAM (NOT TO SCALE) SHOWING TRACK CLEARANCES DURING CONSTRUCTION OF ETHEROW BRIDGE

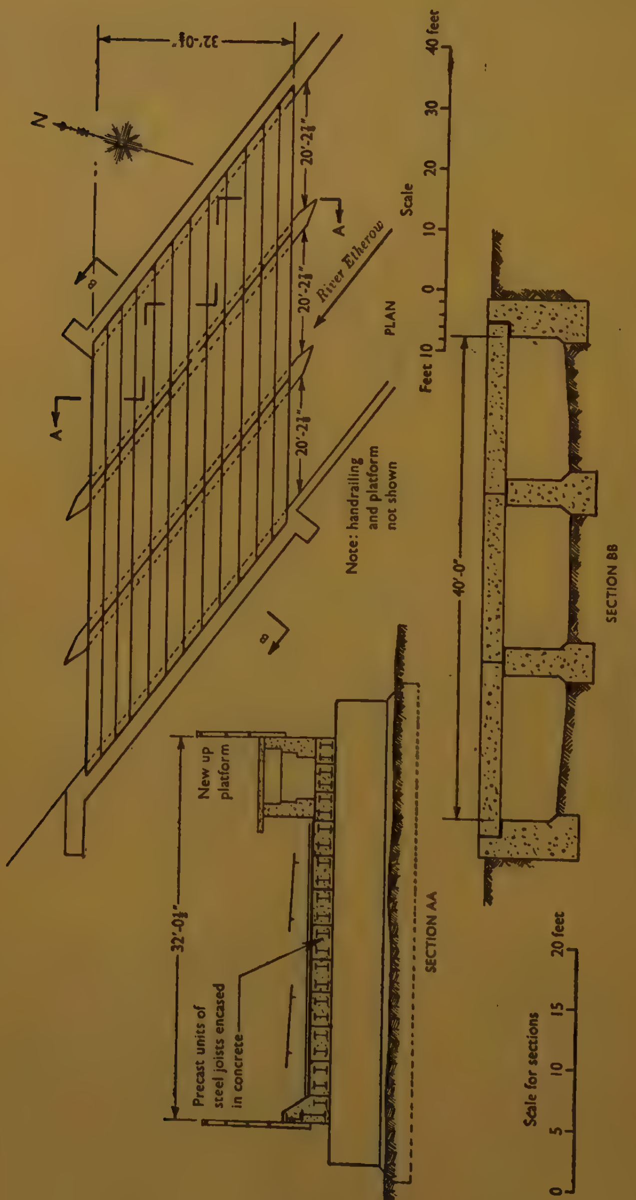
At Dunford Bridge the new portal is on the straight, 100 feet south of the old down-line tunnel, and the new approach converges gradually towards the existing tracks, necessitating a new deep cutting, a new road overline bridge, and certain road and stream diversions as well as station and siding reconstruction. The new road bridge was built with concrete-encased plated girders, built up to 27-by-12-inches, with precast concrete jack-arches set between the lower flanges. (Figs 10, Plate 2.)

TYPE OF CONTRACT

A contract document was prepared on the orthodox lines of a priced bills-of-quantities contract containing 363 items with the usual rise and fall adjustment clause to meet variations in the prices of labour and materials.

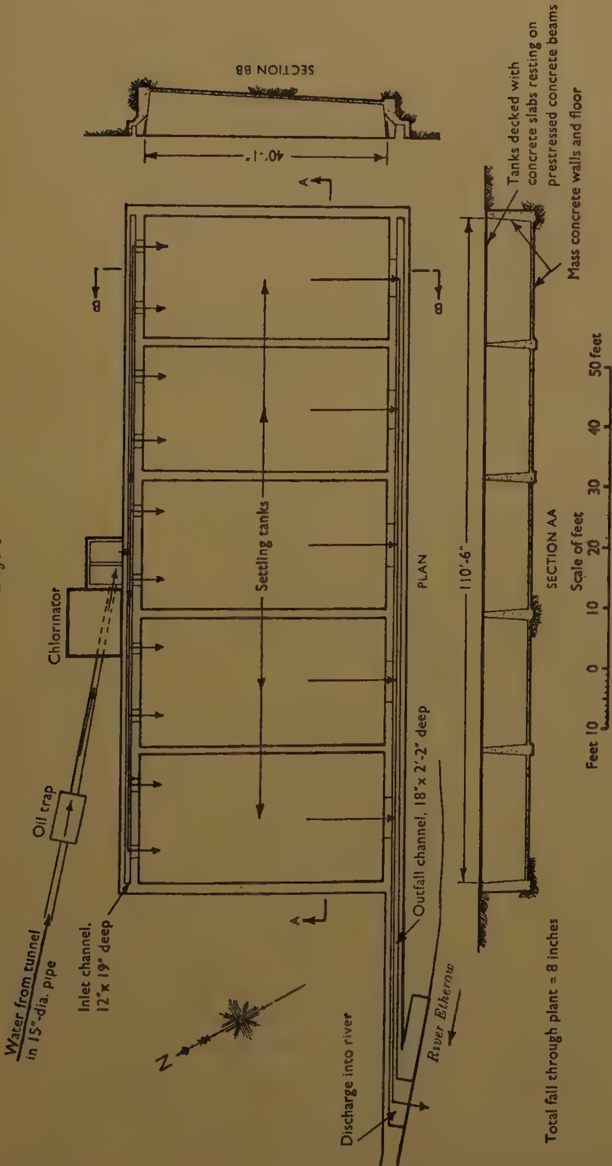
Invitations to tender were issued to seven firms of civil engineering contractors experienced in tunnelling ; of these, two declined to tender, three returned priced tenders as invited, and two submitted the bills-of-quantities priced and totalled as a target. In examining the tenders it seemed apparent that, taking into account the known qualities of the rock to be excavated, and consequent unpredictable risks, and the ruling uncertainties as to labour and materials, contracting firms were unwilling to accept the possible heavy risks unless fully protected by quoting high rates.

Figs 8



River Etherow Bridge. General Arrangement

Figs 9



WATER-TREATMENT PLANT FOR TUNNEL EFFLUENT

In the circumstances it was decided to re-invite tenders on the basis of a target sum represented by the total of the priced bills-of-quantities, and the payment of a fixed fee adjusted by bonus or limited penalty dependent on the actual cost and period of completion—the latter being specified as 42 months. Furthermore, it was specified that the work would be financed through an imprest account maintained in the name of the Railway Executive, from which the contractor would be authorized by the engineers to draw cheques in his own name to recoup his actual outgoings. Messrs Balfour Beatty & Co. Ltd, who had carried out all the recent repair work in the old tunnels, and who therefore had intimate knowledge of local conditions, were awarded the contract on that basis.

The difficulties met during the execution of the work fully justified the departure from the orthodox type of contract, and a most carefully worded contract-deed was drawn up in amplification of the contract document to ensure that the contractor should have every incentive for economical working and early completion of the work. Nevertheless the Authors are of the opinion that the perfect system has yet to be devised. In this type of contract, the Resident Engineer has to give his approval for every purchase made by the contractor, for his methods of operation, the salaries paid, and the bonuses offered ; yet the contractor is responsible for the successful and economical execution of the work. The engineer must, therefore, exercise his powers with the utmost discretion, since refusal to approve the contractor's action and insistence on any variation could, rightly or wrongly, be blamed for lack of progress, or inefficient or uneconomical working. If a contract of this nature should go against the contractor, and he should incur the maximum financial penalty at an early stage, no really effective sanction remains and, since the subsequent cost has no direct effect whatever upon him, the contractor cannot possibly view the expenditure in the same way as the engineer and the employer. It is true that the prolongation of the contract at such a stage involves the contractor in unremunerative head-office expenditure, and in the loss of his key staff from other, possibly more rewarding work, but this may in itself produce a tendency to further expenditure by over-generous staffing and a prodigal use of materials designed to reduce the period of such losses. The position is equally unsatisfactory to the engineer, who likewise has to maintain site staff and who, while remaining responsible to his employer for the early completion of the work, is unable to influence it by any means other than those of persuasion or suggestion. As is the case in most civil engineering works where man is wrestling with nature, the answer lies in mutual understanding between employer, engineer, and contractor. So long as the work is subject to unknown or unpredictable factors, no legal documents, however carefully phrased, can be devised to meet all the contingencies likely to arise, and, accordingly, the operation of the contract has to be adjusted to meet circumstances as they develop.

Construction

The contractor planned to drive a pilot tunnel 12 feet wide and 12 feet high from each portal, and from a central shaft, and thereafter to enlarge the pilot to full section, working only from the two portals. The proposal was that the pilot should be enlarged by radial drilling instead of face drilling. Thus the drilling gangs would work continuously and without interruption from mucking operations, receding from portal to shaft and using the shaft as their source of supplies. Giving the drillers a sufficient lead, the following gangs would charge the holes and blow a length of enlargement which the mucking gang would then remove by way of the portal. Under this system no one operation would interfere with another, since drilling, charging and blasting, and mucking would be practically continuous and the gangs would have unimpeded use of the enlarged tunnel for the removal of the spoil. The system appeared ideal and the targets of 80 feet per week per face for the pilot tunnel, using orthodox face drilling, and 120 feet per week for enlarging by radial drilling seemed modest enough and would have ensured completion within the specified 42 months. Unfortunately, however, in this particular class of rock, radial drilling proved to be completely impracticable and the entire system had to be abandoned. Holes charged radially blew a ragged and uncontrollable section and no method could be devised which produced the approximate profile required without undue overbreak on the one hand, or excessive trimming on the other. Undoubtedly the nature of the shaly rock contributed largely to this disappointing result, but it is doubtful if radial drilling could ever achieve entirely satisfactory results. The classical method of face drilling allows the holes to be disposed so as to remove a central wedge of rock, ease out the bulk of the rock, and finally roughly cut the shape required round the perimeter of the profile. This produces, in good rock, a well-shaped perimeter and a roughly shaped leading face in which the next round is drilled. No such refinements can be expected of the radial system, which subjects the perimeter of the tunnel to the same treatment as face drilling subjects the leading face of a tunnel. In good rock, radial drilling might be expected to cut a well-shaped face but, at best, a doubtful profile. In bad rock it can serve no useful purpose. Such are the conclusions resulting from practice, but these results were not foreseen and the advantages of the successful application of the system would have been so great that it is not considered that apologies are required for the experiment. It would be of great interest to see the results of radial drilling in a close-grained rock, where the disadvantages of a roughly cut profile (and consequent overbreak) might well be outweighed by the resulting economy in time and manpower.

The abandonment of radial drilling created a serious problem of re-organization, since, as that system required only one working shaft, only four working faces were now available for the operation of face drilling.

The possible rate of progress at an enlargement face (such as the one seen in *Fig. 11*, facing p. 526) could not be predicted with any degree of certainty and experience was obtained very slowly. Because of awkward rock conditions at the portals, it was not practicable to start the driving of full-section faces directly from the outside. Instead, break-ups from the pilot tunnel had to be made a little way inside at each end, to form a chamber from which enlargement faces could be worked inwards, while the enlargement back from the break-up position to the portal was excavated very cautiously. This operation of forming break-up chambers wherever required in the pilot tunnel turned out to be a slow and difficult task, and greatly delayed the start of all enlargement faces in poor rock.

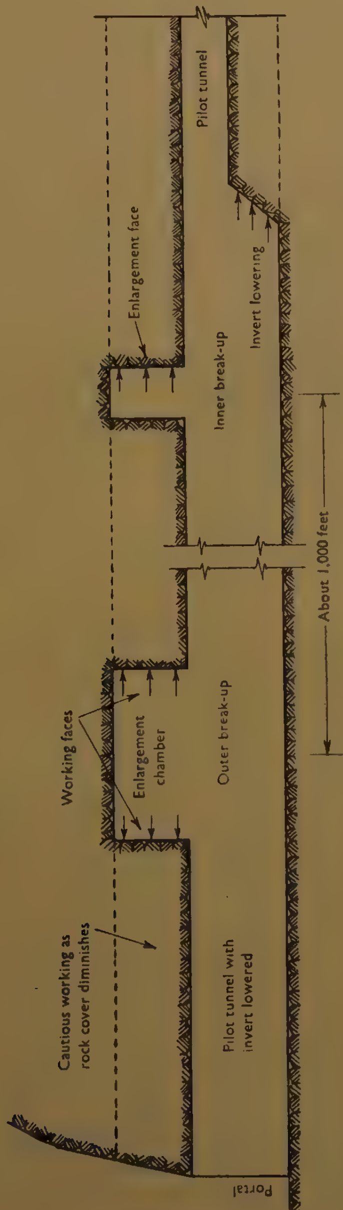
When the enlargement faces near the portals were eventually got under way, the indications were that in the doubtful shale rock forming roughly three-quarters of the length to be driven through, speeds of 20 to 30 feet per week were all that could be expected with the methods adopted at the outset. Working from the four points with straightforward access, this was less than half the speed needed to finish the enlargement in the specified time, and even then, with pairs of faces junctioning half way between the shaft and each portal, concrete lining, which could advance from the portals only, would be held up at the quarter points until the completion of enlargement work. Obviously, the opening up of enlargement faces at additional places was essential, and plans were therefore made to form and work from chambers broken out from the pilot tunnel at points ahead of the main enlargement faces (see *Fig. 12*).

Access to these additional chambers, however, was far from straightforward. First, all traffic and services to them passed through the main face, with consequent interruptions; secondly, since much of the pilot (before the final abandonment of radial drilling) had been driven centrally, the pilot invert was generally 6 feet above the invert of the enlarged tunnel, and before traffic could pass through the main enlargement to the extra face this floor had to be lowered.

The invert lowering was in fact carried out, although where the ground was bad it was a troublesome operation and left stretches of the pilot tunnel in an unstable condition. Work at the inner break-ups, however, turned out to be extremely difficult. Had it been possible to work to a rigid cycle of operations at the inner and outer faces, so that the traffic and services interruptions could have been phased, reasonable progress might have been achieved. With both gangs struggling in difficult ground conditions, without any regular cycle of operations, and with the risk of the inner gang being cut off by a heavy fall at the outer face, the impracticability of the arrangement soon became apparent.

It is interesting to speculate how the main problem of widening the tunnel at a satisfactory rate would have been tackled had not a disastrous roof fall occurred at this stage. The driving of the pilot tunnel had proceeded at the programmed rate of about 80 feet per week and no untoward

Fig. 12



INNER BREAK-UP

incident had occurred. The section was a small one and it had been found, in general, that the sections in sandstone stood without support, whereas in the shale sections steel ribs were required. The number of places requiring support was much greater than anticipated and it was found necessary to space these ribs much more closely than expected. Nevertheless, the driving of the pilot had proceeded fairly uneventfully and, because of the delay in completing the sinking of the shaft, was being followed at an earlier stage than originally planned by the enlargement, when the need for immediate robust support for the 31-foot roof-span in shale was demonstrated in a dramatic way. In June 1951, with a few hours' warning, a 72-foot length of enlarged tunnel near the Woodhead end collapsed, causing a complete blockage which took 6 months to work through. Details of the symptoms of this fall, the sequence of events, and the re-tunnelling operation may be of some value and are set out in the Appendix. Signs of a similar event impending near the Dunford Bridge end were observed about the same time ; in each case the load seemed to develop about 3 weeks after excavation. In the second case, there was time to erect additional supports, and this action did prevent further movement. It was, however, a difficult and dangerous job, and a fatal accident occurred during the work.

After the work of enlarging from the Woodhead end had been brought to a stop by the fall, it became evident that it would be several months before this face could be opened up again, or access obtained through it to the inner break-up which had recently been started. After much consideration it was finally decided to drive a haulage-way clear of the tunnel section round the obstruction to allow work to be resumed beyond the collapse. This was successfully done, and the experience gained encouraged the idea that such haulage-ways, or by-pass tunnels, might be used wherever uninterrupted access was needed to points in the pilot tunnel beyond enlargement faces. Accordingly, the original by-pass round the fall was extended, in the form of a tunnel parallel to the pilot and 50 feet south of it, and connexions across to the pilot were formed wherever required. At Dunford Bridge, another such by-pass tunnel was started 600 feet inside the portal, and access to inner break-ups was obtained through it. In the end, a maximum of nine enlargement faces were worked at one time, using a system involving about 9,500 feet of by-pass tunnel, and the original expedient to deal with the fall disaster had become the solution to the access problem.

The planning of the scheme of by-passes and break-ups was quite a complex operation, depending on estimates of relative progress in by-pass tunnel and in enlargement face, in different types of rock, and on traffic capacities of the single-track by-pass tunnels. The need to ensure uninterrupted progress of concrete lining, working inwards from the portals, also affected the programme of work. In order to plan operations, to follow the varying fortunes of the headings day by day, and constantly to revise

the tactics without losing sight of the main objectives, a diagram was evolved on which the past, present, and future progress of all headings could be plotted. This diagram is reproduced in Figs 13, Plate 2, and a study of it will reveal the rates of progress of the various faces and the way in which the available face gangs were deployed in the efforts to keep ahead of the potential progress of the concrete lining.

Many difficulties were met; progress in driving the by-passes did not come up to expectations and rock falls were encountered even in such small adits, traffic arrangements for the removal of spoil and supply of stores and tools to the face sometimes broke down, but the main problem had been solved and progress was ensured. The system is one which deserves serious consideration in any tunnelling work involving the driving of a long tunnel without intermediate access by shaft or adit. For example, during full-face driving from the two portals of the tunnel, a parallel connecting by-pass tunnel could be driven from each portal, thus enabling full-face tunnelling to be carried on at several pairs of faces simultaneously, by turning off at pre-determined points from the by-passes to the line of the tunnel. The by-pass tunnels would, in fact, be adits, but the adits would be parallel to, instead of (as is more usual) at right angles to, the line of tunnel. Where no pilot tunnel had previously been driven, ventilation of the inner faces would present a problem, but not an insoluble one.

Enlargement of Tunnel

After the floor of the pilot had been excavated to the level of the finished tunnel, face working was started on the enlargement. Various systems were tried, including top heading and bench, crown heading, and full-face working, but finally the majority of the work was done full-face, using a steel gantry running on rails at 25 foot-centres, as illustrated in Figs 14, Plate 2, and *Fig. 15*, facing p. 526. This gantry was designed after it had become apparent that the need for rib supports would be the rule rather than the exception, and the contractors, therefore, incorporated with the drilling platforms a length of carriage designed to handle and erect the steel arch-ribs. In this way ribs were erected close behind the excavation as drilling proceeded. In the greater part of the tunnel the closeness of the drilling and ribbing operations was imperative if falls were to be prevented.

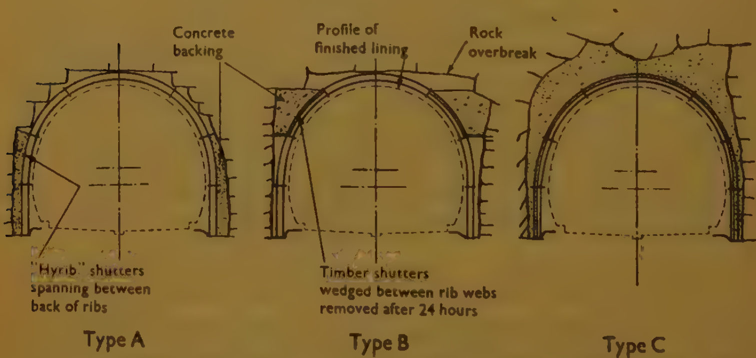
In spite of the help given by the erecting gear on the gantry, ribs had inevitably to be erected hastily and often in bad conditions. As Proctor and White [1] have shown, ribs backed firmly off the rock at a large number of points round an arch are estimated to be stressed to an amount up to 55 per cent less than similar ribs with only a few blocking points, but although the need for good packing was realized, the heavy and awkward overbreak

[1] R. V. Proctor and T. L. White, "Rock Tunnelling with Steel Supports." Youngstown Printing Co., Youngstown, Ohio, U.S.A. 1946.

occurring in much of the shale made such packing exceedingly difficult. Apart from producing an unfavourable load distribution on the rib, the absence of sufficient packing allows small initial movements in the rock which become progressive and throw the dead weight of a great volume of rock, which might otherwise be self-supporting, on to the supports. Nevertheless, when overbreaks up to 6 feet are common, the amount of steel packing soon becomes prohibitive.

The packing problem was partly solved by the placing of a primary concrete filling between the backs of the ribs and the rock itself wherever overbreak and possible rock pressures seemed to justify it. If general improvement to the stability of suspected ribs was required, "Hyrib"

Figs 16



THREE TYPES OF CONCRETE PACKING PLACED BEHIND TUNNEL RIBS

permanent steel shutters were placed behind the rib legs, and concrete walls were formed up to shoulder level. (Type A, *Figs 16*.) If packing out to excessive shoulder overbreak seemed necessary, removable timber shutters were wedged between the rib webs, and the shoulder spaces were concrete-filled. (Type B.) In the worst cases, where a run of shattered rock occurred in the crown, concrete on these timber inter-rib shutters was carried over the roof to form a complete arch of primary lining, sometimes in addition to complete primary sidewalls. (Type C.)

Two points are worth noting regarding this primary concrete. First, in really bad rock it was necessary to concrete right up to the rib nearest the face, and to fire a round very close to green, and sometimes unset, concrete. Secondly, although the placing system could not be anything but an expedient in difficult conditions, and although weak concrete could not be used because of the need to pump, the system represented a gain compared with the filling of these spaces during the placing of the lining itself, since, when excessive overbreak increased the quantity required in the lining beyond a certain point, the whole cycle of lining operations was upset.

Overbreak throughout was extensive. The slabby, jointed, and fissured rock of very variable hardness which was encountered is the worst material imaginable in which to cut a neat hole of any size, and the constant dilemma at the face was to know whether to try to hold loose rock by elaborate packings, or to let it come, in the hope of finding a natural arch above. Very often minor falls before the ribs could be erected left ultimately little choice. Average overbreak, calculated from the concrete volumes put in, was $17\frac{1}{2}$ inches beyond the theoretical excavation line round the walls and arch of the tunnel. This represented 15 per cent of extra excavation and 85 per cent extra concrete in the lining, compared with an overbreak of 10 inches (giving 10 per cent excavation and 48 per cent concrete above theoretical) allowed for in earlier estimates.

Even with all the by-passes in operation and the maximum number of faces being worked, progress was not up to schedule. As between contractor and engineer it is probable that agreement could never be reached on the question of the true causes of poor progress. Bad rock conditions were admittedly the prime cause, an abnormally high labour turnover (400 per cent per annum) with resultant difficulties in maintaining skilled gangs, was a potent influence, but it was felt that the detailed organization and supervision of the work as a whole were contributory factors. The onlooker proverbially sees more of the game than the player and as a contribution to the solution of the problem of improving progress, the engineers instituted a system of time-and-motion studies of all operations. A tunnelling engineer with experience in time studies was given the sole task of analysing every operation involved in the excavation of the tunnel, down to the smallest detail of track lay-out, tunnel lighting, availability of tools, meal breaks, maintenance of plant, bonus systems, etc.; he produced his findings in a series of reports. These reports were made available to the contractor and the implications discussed between the Resident Engineer and the Agent. Many of the factors (such as non-availability of trained men or suitable plant) were ones the contractor was powerless to control, but these time studies did undoubtedly pin-point any weaknesses there might be in detailed organization or supervision, the rectification of which would improve progress, and the system is one which contractors, particularly on large repetitive works, might well adopt to their own advantage.

Excavation Plant and Machinery

Throughout the pilot and enlargement excavation, drilling was done with the light hand-guided type of machine mounted on a single air-operated supporting leg giving a power thrust. Removable cross-bits of $1\frac{5}{8}$ inch diameter with tungsten-carbide inserts were used on hexagonal steels. Drilling speeds were of the order of 9 inches per minute including normal delays, and the drilling as a whole called for less comment in time-study

reports than any other operation. Approximately thirty-two holes were drilled for the pilot face, and fifty-five for the enlargement; the length was generally between 6 and 10 feet, though the number and depth varied a good deal according to the type of rock.

The explosive was Polar Ammon "B" gelignite, with electric gasless delay detonators. In the pilot tunnel a little more than 3 lb. per cubic yard of rock was required in sandstone and about $2\frac{1}{4}$ lb. per cubic yard in shale. For the enlargement it was not possible to obtain separate figures, but the average for the whole job worked out at just less than $2\frac{1}{2}$ lb. per cubic yard.

Mucking for pilots, by-passes, and enlargements was by Eimco rocker shovel (model 21) loading into 2-cubic-yard side-tipping skips on 2-foot-gauge track. For the enlargement, three Eimcos on parallel tracks were worked into the muck pile, and where possible air-driven "cherry picker" hoists were built close behind the face to raise loaded skips and allow empties to be run in behind the Eimcos. Elsewhere, empties had to be shunted at crossings kept as near the face as possible, but this produced more idle loader time than the "cherry picker" system. The Eimcos could generally handle muck faster than it could be got away and though a larger model would have been ordered for the enlargement faces had it been obtainable at the time, the smaller was found to be quite satisfactory.

Battery locomotives were used throughout, except for shunting at the portals and shaft top, where diesels were worked. Artificial ventilation was needed only during the driving of the pilot tunnels.

In supporting the roof of both pilot and enlarged tunnels, altogether about 5,900 tons of steel were used. All the steel supporting the enlarged tunnel was incorporated in the lining. No timber was allowed except where, in relatively few places, steel packing was impracticable.

Tunnel Lining—General Scheme

The contractors had planned originally to start placing the permanent concrete lining at each portal as soon as the enlarging excavation had reached 500 feet. It was then intended to keep pace with the enlarging operation, as carried out by radial drilling, so that the two lining shutters would advance towards the centre of the tunnel at 120 feet per week, and ensure completion about 5 weeks after the end of the excavation.

When radial drilling was abandoned, and the by-pass tunnel and multiple break-up system evolved, various schemes for placing lengths of final lining at inner positions were contemplated. At that time it was not considered that lining could advance at more than 120 feet per week, even with a clear enlarged tunnel to work into, so that the completion date was absolutely governed by the lining progress, which would be subject to a series of checks and delays in the earlier stages as it caught up with the various enlargement faces and waited for them to break through to the

Fig. 11

An Enlargement Face in Shale, partly drilled, Showing Steel Rib Supports in Pilot and Enlargement

Fig. 15

View from Rear of Drilling and Ribbing Gantry at Enlargement Face

Fig. 18

20-foot Length of Sidewall and Arch Shutter Assembled at Woodhead Portal

Fig. 19

100-foot Length of Sidewall and Arch Shutters after Striking. The Clearance from the Concrete is about 4 inches. The Draw-off to the Stope to Existing Shaft No. 5 is on the Right

Fig. 20

Shutter Rail Setting: Precast Blocks ready for clipping to Rails

Fig. 21

100-foot-long Shutter for Base of Sidewalls

Fig. 25

View of Roof Fall of 8 June, 1951. Later the Six Ribs in the Foreground also Collapsed

Fig. 26

View looking up into the Cavity formed by the Roof Falls of June 1951

Fig. 27

SPECIAL GANTRY OF BAILEY BRIDGE UNITS WORKING THROUGH ROOF COLLAPSE REGION

Fig. 28

FRONT OF SPECIAL GANTRY USED AT ROOF FALL, 1951

Fig. 29

NEW PORTAL AND APPROACH CUTTING AT DUNFORD BRIDGE AFTER COMPLETION

Fig. 30

NEW PORTAL AT WOODHEAD AFTER COMPLETION

Fig. 31

Approach to New Portal at Woodhead, showing New and Existing Bridges and Platforms

Fig. 32

Completed Tunnel near the Dunford Bridge End, looking East

Fig. 33

Completed Tunnel at Woodhead End from 600 feet inside the Portal

chamber beyond. Obviously, had it been possible to start lining operations at additional points—for instance, working outwards from the shaft bottom—the pattern of break-ups could have been simplified and the job finished more quickly. In addition, the programme of lining only from the portals inwards meant that in some places 18 months would elapse between excavation to full size and the placing of the lining: all this time the rock would be exposed and the steel supports would have to be watched for signs of distortion and impending failure.

However, there were serious obstacles to the setting-up of extra lining shutters and to the feeding of concrete to them, when the shaft was working to capacity in removing spoil, and the traffic capacity in the by-pass tunnels already represented a bottleneck to excavation. The camp had already been expanded to accommodate 1,100 instead of the anticipated 600 men, merely to supply gangs for the extra break-ups, and a further expansion to allow extra concreting gangs could hardly be contemplated, especially when the peak labour force would have been needed for only a short period.

Finally, the plant had been designed and ordered for the original plan and would have been difficult to modify in the time available. Consequently, all efforts were concentrated upon improving the speed of advance of the shutter system, by lengthening the shutter, stepping up the rate of placing concrete, and cutting down the period of striking, shifting, and re-setting the shutters, without basically changing the system or increasing the planned labour force.

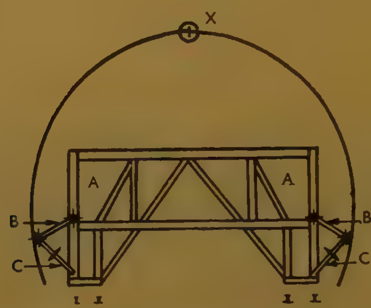
Shutter Setting

The system was designed to permit the concreting of the whole of the sidewalls and arch, in long lengths, in one operation. Consequently a large and heavy shutter was built, capable of supporting about 6-foot thickness of wet concrete for its span of 27 feet, and at the same time being readily movable, easily struck and re-set, and clear of traffic on two 2-foot-gauge tracks in the tunnel centre. It consisted of a braced steel carriage AA (*Fig. 17*) running on bogies on the two outer tracks, from which, along each side, levers BB were pivoted. The skin of the shutter, formed of $\frac{1}{4}$-inch steel plates between 8-by-$14\frac{1}{2}$-inch box girder ribs, was fashioned to the required profile of the tunnel lining and was hinged along the crown point X. It was linked to the carriage only by the levers BB, and a row of screw jacks CC, so that by jacking down, the two halves of the skin moved downwards and inwards, the motion being approximately radially away from the profile. The appearance of a 20-foot length of the shutter when assembled is shown in *Fig. 18* and a 100-foot length set up in the tunnel is seen in *Fig. 19*.

The 100-foot shutter weighed 230 tons, so that the rails it ran upon had to be firmly set. It was necessary to have the carriage correct for line and level, and it followed that the position of the rails determined the setting

of the whole lining. A concrete slab was, therefore, first put down, and into it were set hoops on which centre-lines were scribed by engineers working from survey floor stations. Thirty-foot lengths of 50 lb.-per-yard rail were set from the hoops, and precast concrete blocks were clipped to them (see *Fig. 20*). The rails were re-checked for line and level by both contractors

Fig. 17



SIDEWALL AND ARCH SHUTTER LINKAGE

and resident engineer's staff, and concrete was then placed to the underside level of the rails, thus surrounding all the blocks. The rail-setting tolerance was $\pm \frac{1}{4}$ inch for line and level. After the shutter had passed, the rails could be unclipped, and 30-lb.-per-yard rails placed in their stead, thus providing an additional skip road without tying up more 50 lb. rails than necessary. The footing, together with the lowest 18 inches of the sidewall, was placed after the rail track had been set, using a shutter running on the outer rails as illustrated in *Fig. 21*.

The excavation of the footings, the setting of shutter rails, and the concreting of the footing and haunch of the lining had all to be carried out between the excavation to full size and the concreting of the main lining, but these jobs could all be speeded up more or less without limit. It was the placing of the sidewalls and arch concrete which set the pace, and the lining organization centered round this process.

CONCRETE MATERIALS AND PLACING

From what was known about the rock to be excavated, it had not been expected that the tunnel spoil, or any crushed local rock, would be suitable for concrete aggregates. However, the subject was re-examined when the contract started, and a number of tests were made on the best of the sandstones and gritstones found locally. The conclusion was that the rock, after selection, crushing, and screening in any plant that could be visualized as being economic, was too variable and weakly cemented to be

used in medium-quality concrete. Accordingly, the very large quantities required (amounting to 97,000 tons of sand and 142,000 tons of coarse aggregate) had to be brought from a considerable distance. Haulage charges rose to such an extent between the decision to use outside sources (1950) and the delivery of the bulk of the material (1952/53) that the economics of local production compared with transport were appreciably changed, but even if these increases could have been foreseen, and the low strength of the aggregate particles accepted, it is doubtful whether production on the scale required could have been undertaken.

The possibility of using special cement in part of the lining concrete in order to resist the action of sulphurous smoke fumes from locomotives had been examined, but the phasing of the electrification programme to eliminate steam working in the new tunnel made this unnecessary, and ordinary Portland cement was used throughout. Ground-water samples from the tunnel were analysed and found to contain no substances liable to harm Portland-cement concrete. Cement supplies in sufficient quantity, and of a high and consistent quality were available.

The materials were weighed into batches just outside each portal, and were mixed and pumped by a battery of machines mounted on the shutter tracks and kept as close as possible to the shutters. Transport from the batchers to the mixers was by trains of two bogies (each bogie carrying 6 batch hoppers) hauled by battery locomotives on the 2-foot-gauge track; to keep the batch hoppers to a suitable size, and to ensure that the mixers did not foul the through traffic, the batch size was limited to ½ cubic yard. This proved to be undesirably small in relation to the volumes of up to 1,000 cubic yards which had to be placed in one operation in the minimum time.

To get the concrete from the mixers to the shutter, the only practicable alternatives were pumps or pneumatic placers. At that time the engineers had had considerable experience with a tunnel-lining in Scotland in which placers were being used.[2, 3] In the comparatively small and thinly lined Fasnakyle tunnel, the placers gave qualified satisfaction, but it was feared that with the dimensions of Woodhead, and in particular the greater height, difficulties of segregation might be intensified. Concern was felt about the effect of the violent discharges of concrete from placers, since in many regions ribs which supported loose rock and packings had little stability against forces along the line of the tunnel once the temporary timber spacers were removed. The possibility that rib movements induced during concreting might allow rock and packings to fall on to the back of the shutter counted against the idea of placers. There were also fears that extra workability, demanding the addition of cement to the mix, might

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[2] C. M. Roberts, "Special Features of the Affric Hydro-Electric Scheme (Scotland)." Proc. Instn Civ. Engrs, Pt. I, vol. 2, p. 520 (Sept. 1953).

[3] E. C. Dillon, "The Mullardoch-Fasnakyle-Affric Tunnels." Works Construction Paper No. 16, Instn Civ. Engrs, 1949.

prove essential with the use of placers. None of these anxieties, however, would have prevented at least a trial with placers, since their characteristics of high output, simple operation, and low maintenance were very necessary, and it seemed likely that methods of overcoming the objections could be worked out. It is known in fact that tunnels of comparable size in India and America have been lined with placers in recent years. However, it proved impossible to obtain placers in the time required and with the currency available.

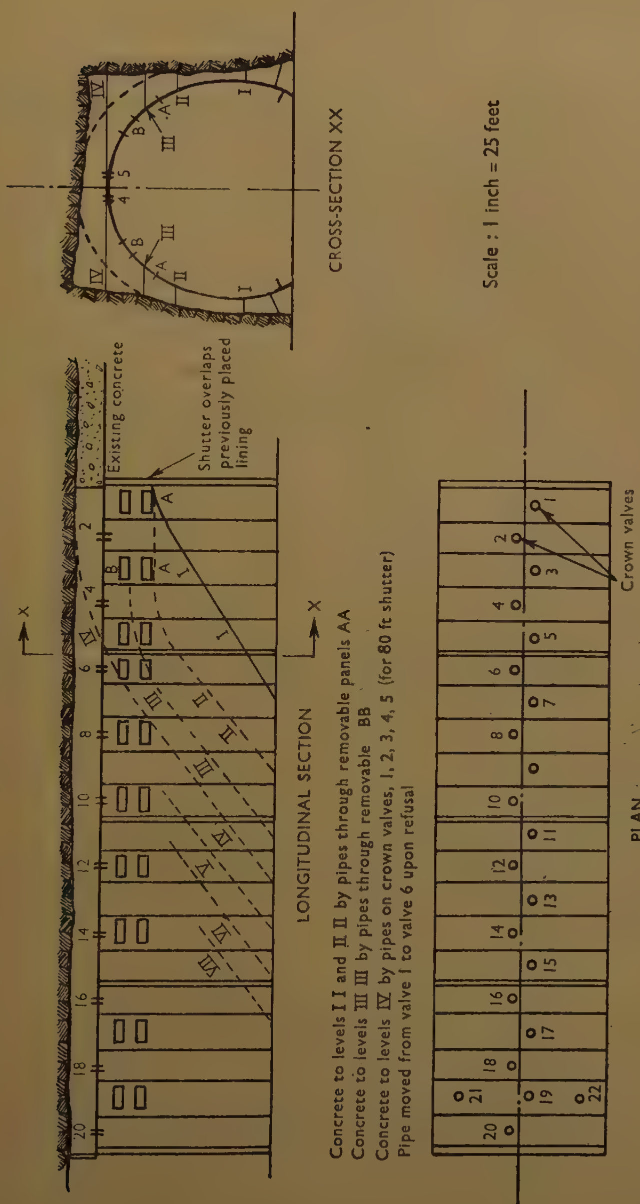
The choice was therefore narrowed to 6-inch-diameter or 4-inch-diameter pumps. The former could deal with 1½-inch aggregate and had more than double the output of the latter per unit, but the capital cost of the number of smaller pumps giving the same delivery was much less, and the extra cost of changing to ¾-inch-maximum-size aggregate turned out to be small. Smaller units were also favoured for the additional reason that the breakdown of one unit would have a less disastrous effect on the cycle of operations.

The 4-inch pumps were thus decided upon, but their peak output was rated at 10 cubic yards per hour, and this rate had to be compared with the very large areas in plan over which concrete would have to be placed when filling 100-foot lengths of shutter with high overbreak. The rate of placing concrete affected not only the progress but the monolithic nature of the lining, since with the level rising slowly, any breakdown or maldistribution was likely to result in either unintentional construction joints, or in the disturbance of partially set concrete. The maximum number of pumps which the batching and transporting arrangements could feed was, therefore, installed; six were coupled to the 100-foot shutter at the east end, and five to the 80-foot shutter worked from the west.

Had the concrete surfaces during the placing in the sidewalls and arch been kept approximately horizontal, even the maximum pump output would have been insufficient to maintain an adequate rate of rise, especially at shoulder level, where overbreak was nearly always very large. A sloping flow method was therefore adopted (see *Figs 22*) in which concrete was built up as quickly as possible to crown level at the back of the shutter, against the stunt end of the last length completed, and thereafter all pumps were connected to valves in the crown, with the concrete flowing forwards and downwards to complete the sidewalls up to the forward stunt end. The pumps were connected to valves as far back as possible and the pipes were moved forward only when the resistance to further concrete being forced in became excessive. Concrete was thus squeezed between the packings and ribs under the pump pressures, and such pressures were at times sufficient to buckle the shutter ribs and skin plates in the crown and shoulders.

Clearly the success of this system depends upon a high level of supervision and inspection, and on the conscientiousness and skill of a large number of operators. The uniformity and consistency of the mix, and the

Figs 22



METHOD OF CONCRETING SIDEWALL AND ARCH OF TUNNEL LINING

scheme of control and testing applied are also of great importance; the
will be the subject of a separate Paper.

Results of the Adopted Method of Lining

Heavy demands were made on detailed works planning and mainte
ance by the intricate interlocking system made necessary, in which a
hold-up at stockpiles, batchers, tunnel trackwork, hoists, mixers, remixe
pumps, and pipelines could throw the whole operation out of gear. Ea
part of the system worked reasonably well, but at first breakdown
derailments, and local disorganizations cut down the pump outputs
below 30 per cent of their maximum, and jeopardized quality as well
progress. Organization gradually improved, and many of the men becan
proficient (though the high labour turnover resulted in the continu
breaking-up of gangs), and by the time the excavation was sufficient
advanced to allow the lining to proceed without interruption, progre
exceeded expectations. At best, it was possible to concrete three lengt
(of 80 or 100 feet) on alternate weeks, and two lengths intermediately,
the timetable detailed in Table 1.

Striking, moving, and re-setting times were kept low (and sometim
under the 12 hours allowed for) except where water had to be divert
extensively after the shutter was moved forward, or where rib distortio
necessitated cutting back steel and rock at the last moment, but the actu
concreting times, as represented by the 24-hour average concreting perio
seldom represented maximum pump output. The factor limiting t
average rate of pumping was the rate of feeding the mixers with d
batches, and this in its turn was dependent upon the speed of shunti
bogies and hoisting hoppers. The skill needed to handle these operatio
continuously for 24 hours at maximum speed was usually beyond t
powers of the labour available.

As an overall measure of the final rate of progress, it may be record
that for the last 7 months of lining operations the average cycle was 2·
moves of each shutter per week, representing totals of 392 linear feet p
week or 3,590 cubic yards per week.

Grouting

The need for preventing initial movement in the rock, distributing t
rock load on to the arch, and providing contact between rock and st
supports below shoulder level where passive pressures might be call
into play, had been made very clear by the behaviour of the arch duri
the temporary support period. In spite of the pressure used to comp
the arch concrete and to force it between the packings and against t
rock, there were many places in shale sections where rib supports w
closely spaced and packings were dense and at which, in consequen
complete penetration seemed problematical. Wherever there seem

TABLE 1.—CONCRETING TIME-TABLE

		Week 1	Week 2
Sunday			
Monday	Midnight	Start	Strike, move, and reset (12 hours)
	8 a.m.	Concrete (24 hours)	Start
	Midday		
	4 p.m.		
	Midnight	Finish	Concrete (24 hours)
Tuesday	8 a.m.	Setting	
	Midday		Finish
	4 p.m.	time (24 hours)	
	Midnight	Strike, move, and reset (12 hours)	Setting
Wednesday	8 a.m.		time (24 hours)
	Midday	Start	
	4 p.m.		Strike, move, and reset
	Midnight	Concrete (24 hours)	Start
Thursday	8 a.m.		
	Midday	Finish	Concrete (24 hours)
	4 p.m.	Setting	
	Midnight		Finish
Friday		time (24 hours)	
	8 a.m.		Setting
	Midday	Strike, move, and reset (12 hours)	
	4 p.m.		time (24 hours)
	Midnight	Start	Strike, move, and reset (12 hours)
Saturday	8 a.m.	Concrete	
	Midday	(24 hours)	Maintenance (all gangs) (4 hours)
	4 p.m.	8 hours overtime	
	Midnight	Finish	Maintenance
Sunday	8 a.m.	Setting	(as required)
	Midday		
	4 p.m.	time (24 hours)	
	Midnight		

doubt about the filling of voids, grout pipes leading to the worst or highest points were fixed to the back of the shutter and cast into the lining, and of the 16,037 feet of the tunnel, about 12,500 feet had pipes cast-in, at intervals varying between 1 pipe per yard and 1 per 10 feet.

Grouting was carried out with pans of 8 cubic yards capacity, using a maximum pressure of 50 lb. per square inch. Nearly all the grout accepted went in with little or no pressure resistance, which suggested that voids,

rather than rock fissures, were being filled. The average cement acceptance over the 12,500 feet grouted was approximately 8 cwt per foot run of tunnel, which would be equivalent to a skin about 4 inches thick over the back of the arch and shoulders.

Rock Falls: Remedies and Preventive Methods

The first roof fall was the most extensive, but a second major collapse occurred when the last few feet of enlargement was being excavated. In addition, on several occasions there was a run of shattered rock from the crown or shoulders leaving cavities of up to 25 feet outside the tunnel area, and in two regions rock-pressures distorted the original rib supports until they fouled the inner concrete profile, so that the heavy thrusts had to be transferred while steel and rock were cut out and new supports erected in small sections.

Particularly after the first roof fall, the question of temporary support was predominant in consideration of excavation methods. Steel ribs had been erected close behind the enlargement faces even before the fall, but absolute safety would have demanded much heavier sections at much closer centres than were at all economic or practicable. On the other hand, it was quite evident that few liberties could be taken in this type of work, and the risk of failure could be kept within reasonable bounds only by constant examination of the roof, and re-estimation of the rib spacing and section. This was a heavy and constant responsibility which ended only when the concrete lining was completed.

The erection of ribs was in itself often hazardous, especially when footings had to be prepared under weak unsupported shoulders. Any method of reducing the instability of the rock around the arch would have been most desirable, and consideration was given to the possibility of grouting rock at this level ahead of the enlargement. A general compaction might have been achieved in loose shattered rock, and water movement might have been directed away from the weakest points. The uncertainty of getting the grout to go where it was required in this type of ground, in which bedding, fissuring, and jointing is so marked, together with the difficulty of organizing a gang to grout the rock ahead of the face, and the fear that face blasting would undo much of the effect of the grouting before its usefulness was felt, combined to influence the decision not to give a trial to this scheme.

Most of the classical tunnelling methods, such as top heading, the crown and bench method, fore poling, etc., were considered, and some were tried. In sandstone which did not need temporary support, the best rate of progress with the enlargement face (about 100 feet per week) was achieved with a crown heading and benches 70 feet long, but generally speaking, in ground requiring any support, the full enlargement face with complete ribs erected immediately after excavation, and packed to the

rock with concrete when necessary, was the standard method eventually adopted. As noted above, however, the concrete packing, or primary arch, survived nearby blasting so unexpectedly well even when green or unset, that the possibility suggested itself of keeping a complete primary concrete arch close to a face, perhaps with a hood shuttered on polings cantilevered forward from removable ribs. Compression stresses in an arch in full contact with the rock are not high, and had such a scheme proved workable, the saving in steel, overbreak, and labour would have been considerable. It was not, however, tried, and the advantages and drawbacks remain speculative.

Another possibility considered was the reduction or prevention of the deterioration of the shale exposed for long periods between excavation and lining by "guniting" the surface. The introduction of this additional process would, however, have slowed up progress and would have been very costly, and the system was not introduced. In fact, few, if any, falls occurred which guniting could have prevented and little advantage would have been gained by its adoption.

Special Conditions

A number of special conditions, some natural and others imposed, complicated the driving of this tunnel in varying degrees. Perhaps the most important was the fact, already mentioned, that the site, whilst allowing two points of entry (the portals) was unsuitable for intermediate entry by adits or by shafts of any depth less than about 500 feet.

The total amount of ground-water entering the tunnel before it was lined varied greatly with rainfall but, generally, did not exceed about 100 gallons per minute. A temporary peak of more than 10 times this flow occurred during the driving of the pilot tunnel, when a saturated band of sandstone overlying shale was first pierced, but this flow subsided in a few days.

The trouble caused by water entries was out of all proportion to the amount to be dealt with, because of the softening effect on the shale, and the difficulty of diverting the seepage from extensive areas away from the concrete during the lining operation. The general policy was to provide a free passage, sealed off from concrete during placing, for all ground-water, conveying it either to the centre drain, to a special pipe cast in the haunch, or (for small amounts) through weep pipes just above ballast level. These water passages, formed by fixing bent sheet-metal, waterproof paper, or pipes against the rock just before setting the sidewall and arch shutter, were kept to the minimum, and did not seriously interrupt the general full contact between concrete and rock. During grouting, injection was stopped if grout broke through into the water channels. The total flow into the tunnel after lining and grouting appeared to be rather reduced.

Next in importance was the disposal of spoil. The tip site at the shaft

was only a few yards distant, but the 467-foot vertical lift made the operation slow and costly. At both portals all convenient sites for tips had either already been used during the construction of the original tunnels or were unacceptable for aesthetic reasons. In addition, the method of driving, whereby about 70 per cent of the excavated rock had to come through the two portals, resulted in very big tips being required in their vicinity. At the Dunford Bridge portal, a deep ravine formed by the Smallden Clough was used and the stream was diverted and culverted to allow the valley to be filled in. At the Woodhead end side-tipping had to be adopted on one slope, and in order to render the tips acceptable to the various authorities concerned, the bulk of the material from this end had to be transported 1½ mile up the valley. The spoil was tipped from tunnel skips near each portal, and transported to the tip sites by dumpers or lorries.

In addition to stipulations about the location of the spoil-sites, various landowners and authorities charged with the duty of preserving the amenities of the countryside concerned themselves with the appearance of the surfaces presented by the completed tips.

Owing to the risk of erosion of soil and on account of the scarcity of suitable "top spit," soiling was carried out completely only in certain cases on exposed slopes, that is, those generally visible to the public.

Enough "fines" were available from the disintegrated sandstone and argillaceous shale upon the top surfaces of the tips and on part of the slopes, to obviate the necessity of providing any soil. Where this was required, however, it was "borrowed" from adjacent sites, or was obtained from a heap formed during stripping of actual tip sites. At those slopes not visible from any road or pathway, soil was spread at random, with a view to lodgement taking place in the crevices between tipped rocks, this providing some hold for subsequent growth of vegetation. Red Fescue S.59 grass seed was selected for use; this was sown at the rate of ⅓ ounce per square yard. With this was mixed superphosphate at the rate of 2 ounces per square yard, and also sufficient quantity of dry sand, for the purpose of indicating the extent of area sown, and also to camouflage the seed. After germination, nitro-chalk was added at the rate of ½ ounce per square yard.

It is interesting to note that seed sown on the 9th October, 1953, had germinated, and minute blades of grass were visible within 14 days.

The proximity of the new tunnel to the existing down-line tunnel, which was heavily used, necessitated elaborate precautions during blasting. It was not known what effect normal blasting would have on the old tunnel and, although site tests by I.C.I. explosives engineers indicated that no harm would result, it was considered essential in the interests of public safety to institute a system of inspection after each blast. Telephones were accordingly installed at each blasting face, and at points in the old down tunnel approximately opposite the blasting faces, and all were

connected to a switchboard in the Dunford Bridge signal box. The foreman in charge of blasting notified the signalman when he was ready to fire; the signalman gave the foreman the "All Clear" as soon as a train had cleared the tunnel, and thereafter allowed no further traffic until a railway patrolman, stationed in the old tunnel, had inspected the line and reported back to the signalman that there was no obstruction. Since the tunnels are more than 3 miles long and the old tunnel is normally full of smoke, and since trains pass through in the down direction at a frequency of roughly eighty per 24 hours, the possibilities of delays to both railway and tunnel work were obvious. In practice, delays were almost negligible, a fact which testifies to the high degree of interest and co-operative spirit existing between the railway staff and the contractor's men concerned. No obstruction was ever found after blasting, and towards the end of the contract the precautions were somewhat relaxed.

The site of the tunnel is remote and exposed, and a camp had to be built to house more than 1,100 workmen, as well as most of the engineers and office staffs. The usual recreational facilities of clubs, a cinema, and canteens were provided and a new post-mark came into use since the dry canteen was, in addition, the "Dunford Bridge Camp" post office.

A new sewage works for this large community had to be built and considerable trouble was taken, at the request of the local authority, to make it suitable for the permanent needs of the small hamlet at Dunford Bridge, which, up to that time, had lacked such an amenity.

Survey

With two straight tunnels only 100 feet away and running parallel to the line of the projected tunnel, there were available two base-lines, more than 3 miles long, for setting out the new work. Nevertheless, the traffic was so dense in those tunnels and the air so seldom free from smoke even when traffic was suspended for a day, that it was well-nigh impossible to obtain a through-sight from one end to the other.

Consequently the azimuth of the new tunnel could be obtained from sights in the old to an accuracy suitable only for preliminary setting-out, and the shaft site could be fixed only approximately from the nearby old shaft. For driving, sinking, and lining a full triangulation was carried out, linked to two convenient tertiary Ordnance Survey stations, the distance between which was calculated from data kindly supplied by the Director, Ordnance Survey.

The portals of the old down tunnel were linked to this triangulation, and the length of the old tunnel (about 15,906 feet) was measured on two occasions, although the operation had each time to be completed within 6 hours. The agreement between the measured length and the Ordnance Survey stations was satisfactory, and the junction errors revealed at the pilot tunnel break-throughs were negligible. Accurate survey work had to

continue to the final completion of lining, since the original floor stations in the pilot tunnel were systematically destroyed as enlargement progressed. The need for an accurate survey in the pilots was not obvious at the time of driving, as it was expected that any error could have been straightened out on junctioning and before enlargement and lining started. Accuracy, however, was insisted on as a matter of good practice, and when the tunnel became split up into virtually isolated sections as a result of the fall and the adoption of the multiple break-up system, this decision was more than justified. The by-pass tunnels naturally added to the amount and difficulty of the survey work.

The main survey work was successful, but difficulties did occur from time to time at the stage where information had to be passed on to foremen and shift bosses. Such setting-out troubles as there were occurred at this point, and were partly at least attributable to the large labour turnover, which affected supervisory grades as well as labourers.

Labour Conditions

The somewhat limited social amenities which it was practicable to provide in this exposed site, with several large centres of population within a radius of 25 miles, made it particularly hard to attract suitable labour. The dangerous nature of the work and the uncomfortable conditions underground added to the disadvantages and, with the fixed wage-structure in Great Britain, generous incentive bonuses were necessary. Two-shift working was adopted and the men were therefore guaranteed a minimum amount of overtime which, combined with a production bonus, made the job attractive. Despite this, the annual labour turnover, as already stated, amounted to about 400 per cent. Undoubtedly men cannot give of their best for 12 hours a day, but had working hours been reduced by operating a 3-shift programme, it would have been necessary to increase the labour strength by 50 per cent and the camp and facilities proportionately. These practical considerations ruled out the 8-hour shift and resulted in individual earnings being out of proportion to the results achieved. This in itself added to the contractor's difficulties, since the high earnings, paradoxically, increased the labour turnover. There is no doubt that many men of the wrong type were attracted by the high earnings and used the Woodhead contract as a staging point on their journeys from one job of a more attractive nature to another. This, in its turn, led to disorganization of trained gangs and an unsettling influence on the steadier types of workmen.

Programme, Progress, and Costs

Construction started in February 1949, with a target completion period of 42 months. The works were completed in October 1953 at a total cost

of approximately £4¼ million, of which just less than £1 million was agreed as being extra cost from the rises in wages and materials compared with the rates ruling in 1948. Some views of the completed works are shown in *Figs 29* to *33*.

Acknowledgements

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The Consulting Engineers to the Railway Executive, Eastern Region, were Sir William Halcrow and Partners, and the Contractors were Balfour Beatty & Co. Ltd.

The Resident Engineer for the Consulting Engineers was Mr J. D. Dempster, with Mr F. A. Sharman as his Chief Assistant. For the contractors, Mr W. Brown, M.I.C.E., was the Agent after the preparatory works were completed until the start of concrete lining. He was succeeded by Mr A. B. Sharp, M.I.C.E.

The Paper is accompanied by fifteen photographs and seventeen sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared, and by the following Appendix.

Appendix

Details of the Roof Fall of June 1951

Working inwards from the Woodhead portal of the new tunnel, after approximately 800 feet, the rock penetrated begins to change from hard blocky gritstones and sandstones to black argillaceous shale. The upper boundary of the gritstones dips at about 8 degrees eastwards, so that on the full tunnel cross-section, shale appears in the roof at Chainage 800, and surrounds the whole section at Ch.930, measuring in from the portal. This region was penetrated by the pilot tunnel in December 1949–January 1950, and the relative positions of the shale boundary, the pilot tunnel, and the full section are indicated in *Fig. 23*.

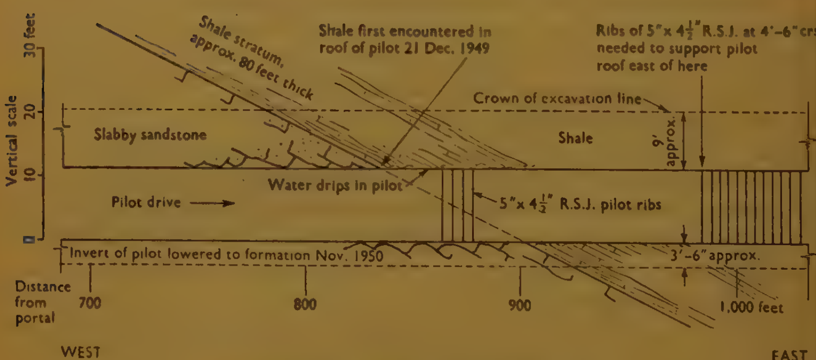
The pilot tunnel (of roughly 12-by-12-foot section) had a shale roof eastward from Ch.860, and a little water came in near the junction, but practically no support was needed west of Ch.1,000, and the pilot tunnel up to this point stood without recorded roof falls for about 18 months. East of Ch.1,000, for about 350 feet, the pilot tunnel was unstable, and many minor falls of rock from the roof were recorded during and after driving. Rib supports at 4-foot-6-inch centres had to be erected as the drive proceeded. There was a good deal of water in the joints and fissures of the shale in this region. The floor of the pilot tunnel was lowered by about 3 feet 6 inches in November 1950 to the formation level of the full section.

Enlargement to full section was carried out from a face advancing eastwards from the sandstone region in the summer of 1951. The erection of 8-by-5-inch B.S.B. ribs at 6-foot centres to support the full section had to be started at Ch.700 and continued at this spacing, but with an increasing amount of packing in the crown. The appearance of these ribs and packings at Ch.880 is clearly seen in the foreground of *Fig. 25*.

The ribs were erected closely behind the face, which was generally advanced in 6-foot rounds. The amount of packing done in the shoulders can be judged from the photograph.

The dates of excavation to full size of the region affected by the fall are indicated in *Figs 24*. On the 4th June, 1951 it was seen that some of the ribs, particularly those around Ch.930, were bearing considerable load, and deflexions were observed in places. Some additional packings were put in and the movements appeared to cease. However, on the morning of the 8th June, deflexions in the steelwork began to increase visibly, and pieces of rock began to drop between the packings and ribs. The ribs themselves began to twist in the crown, and it was clear that the whole support system was being overstrained. The erection of additional main supporting members was judged to be too dangerous, and the drilling gantry and all men were withdrawn about 150 feet from the face. Steelwork deflexions, rock movement, and small falls increased during the day in the region Ch.900 to 976, and at 11.20 p.m. the whole roof of this length collapsed, the twelve ribs supporting it failing generally in the manner shown in *Fig. 25*. A steady trickle of water came from the cavity.

Fig. 23



Longitudinal Section at Site of Fall

The ribs immediately west of the collapse appeared in danger, and remedial work was started by erecting additional ribs working eastwards from Ch.855. A start was made with a concrete bulkhead at the foot of the pile of debris, but on the 24th June, the six ribs west of the fall (that is, all those in the foreground of *Fig. 25*) failed, so that the collapsed section was then 100 feet long. The approximate section of the void created is indicated on *Fig. 24* and the appearance of the rock faces of the cavity is shown in *Fig. 26*, which is a photograph looking upwards from half-way up the debris pile after the second collapse.

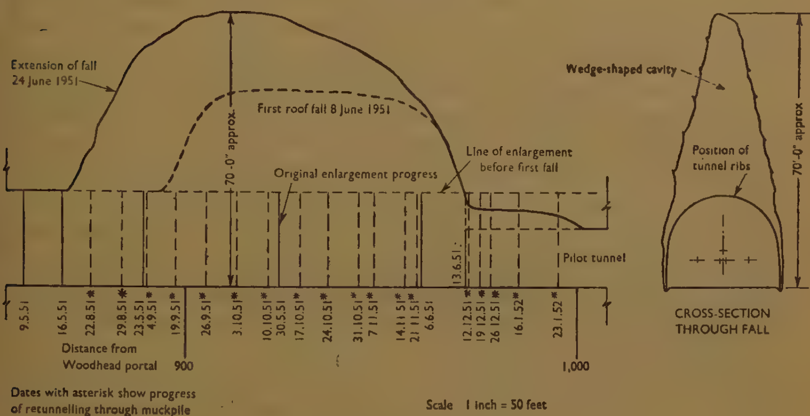
Any further extension westwards of the fall was prevented by the immediate construction of a massive concrete bulkhead. The removal of the steep, high, and unstable pile of debris under the wet and unstable cavity formed was a serious problem, and at first it was thought that it would be necessary to fill the cavity above the debris with concrete introduced through borings from the surface, which at this point is about 200 feet above the crown of the tunnel. Drilling was actually started but a second scheme, of working forward under protection from a hole left in the bulkhead, proved successful.

An initial protective hood to shield men working at the debris pile, was carried on the front of Bailey-bridge units, which were at first cantilevered out from a hole in the bulkhead, and later supported on tracks. The frameworks are seen in *Figs 27* and *28*. As soon as the sides of the tunnel at invert level were clear, 8-by-5-inch B.S.B. ribs, spaced at 12-inch centres, were erected in groups of four, and " Hyrib " steel shutters were inserted between the backs of the ribs to span across the 7-inch gaps

between the flanges. Concrete was then pumped through pipes in the crown, to give an arch not less than 3 feet thick for immediate protection.

No serious setbacks were encountered, though the work was slow and somewhat dangerous. The erection of ribs and the primary arch of concrete was completed in

Figs 24



LIMITS OF ROOF FALL

December, and it was subsequently decided not to attempt to remove the closely spaced ribs or to fill the cavity still left above the primary concrete, but to concrete through the fall region in the ordinary way with the lining shutters using an extra workable mix to ensure that the ribs were fully encased. Thus, although there was no contact between lining and rock over a short span of the crown, an arch of reinforced concrete not less than 5 feet thick was formed under the void. The final lining was completed in 1952, and on inspection in November 1953, no difference could be seen between this portion of the tunnel and any other.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides. To supplement the slides a short section of a film was shown later in the discussion. The film was introduced by Mr F. A. Sharman.

Mr John Ratter said that the story of the new Woodhead tunnel was one of a changing battle against natural forces rather than a methodical progress along preconceived lines.

There had been two main changes of plan during its construction. The first had been the abandonment of radial for face drilling, and the second, the adoption of the by-pass tunnel which, though originally inspired by the very big rock fall at the Woodhead end of the tunnel, had eventually become an essential feature of the successful completion of the job. Thus the final solution at Woodhead was not at all what had been originally

planned. Mr Scott had already said in his introductory remarks that, had the conditions encountered been known beforehand, the single double-line tunnel would still have been preferred to two single-line tunnels. If the Authors had known what the rock conditions were like, would the pilot tunnel have been driven in the same manner? Obviously it would have been driven at a different level, but would it have been constructed in the middle of the tunnel and would the by-pass tunnel not have paid a greater dividend if it had been used in the first instance as a pilot?

Referring to the question of labour in the tunnel, Mr Ratter said it was an extraordinary fact that the turnover had been 400 per cent per annum, so that the average time spent on the job per man had been about 3 months. Paradoxically, apparently one of the reasons why the labour turnover had been so rapid had been the high wages—presumably the men came to get all they could in a short time and then went somewhere else where work was easier. There had been an element of danger in the work which had probably not been very encouraging to the labourers, but one could imagine the difficulties of the contractor with such a rapid labour turnover.

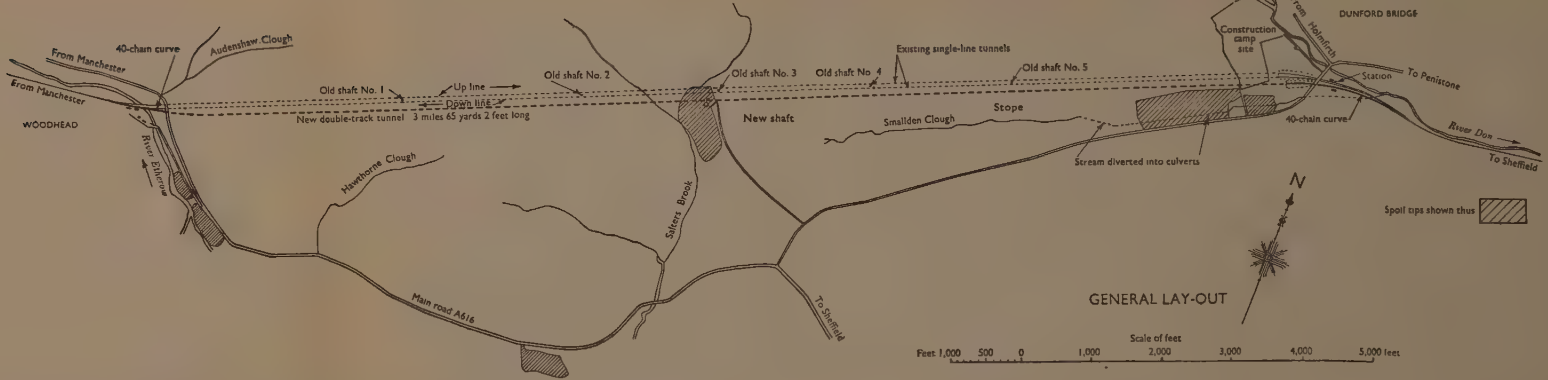
The question of the contract document was mentioned in the Paper, and also the difficulty experienced in obtaining a suitable form of contract for such a job where the conditions to be encountered were unpredictable. Under present-day conditions with costs sky-high, nobody wanted to enter into a gamble, yet the very nature of tunnel construction of that kind was a gamble. Hundreds of thousands of pounds had been at stake depending on the conditions found in driving the tunnel. The original contract document had been well thought out and had conditions inside the tunnel been a little better, it would have been perhaps the best that could have been devised; but there was no way of tying down a contract document to take into consideration circumstances that could not be foreseen. It was a question of the three main parties concerned pulling together with understanding of the difficulties.

It was well known that, in tunnel work, transport was all-important. Tunnelling was mainly a matter of transporting rock from one place to another and of getting concrete from the mixer to the rock face. Provided that the difficulties at the face could be overcome, then transport was the key to the situation. Did the Authors consider that the transport system used had been the best possible? Could conveyor belts have been used successfully or had the system employed been the best that modern equipment could provide?

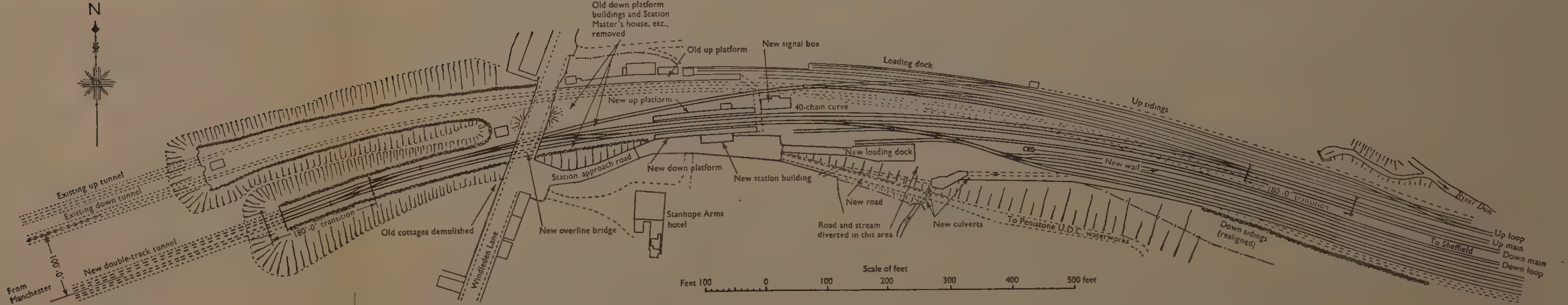
Sir **Andrew MacTaggart**, referring to the question of the form of contract, said his firm had had a great deal of experience with that form of contract, and his experience was that, so far as the principals were concerned, there was never any difficulty in operating a contract of that sort. Relationships between the employers' and the contractors' representatives on the site could be rather difficult at times, due to personalities, but he was very glad to say that, although they had sometimes had different

WOODHEAD NEW TUNNEL: CONSTRUCTION OF A THREE-MILE MAIN DOUBLE-LINE RAILWAY TUNNEL

FIGS 5



GENERAL LAY-OUT



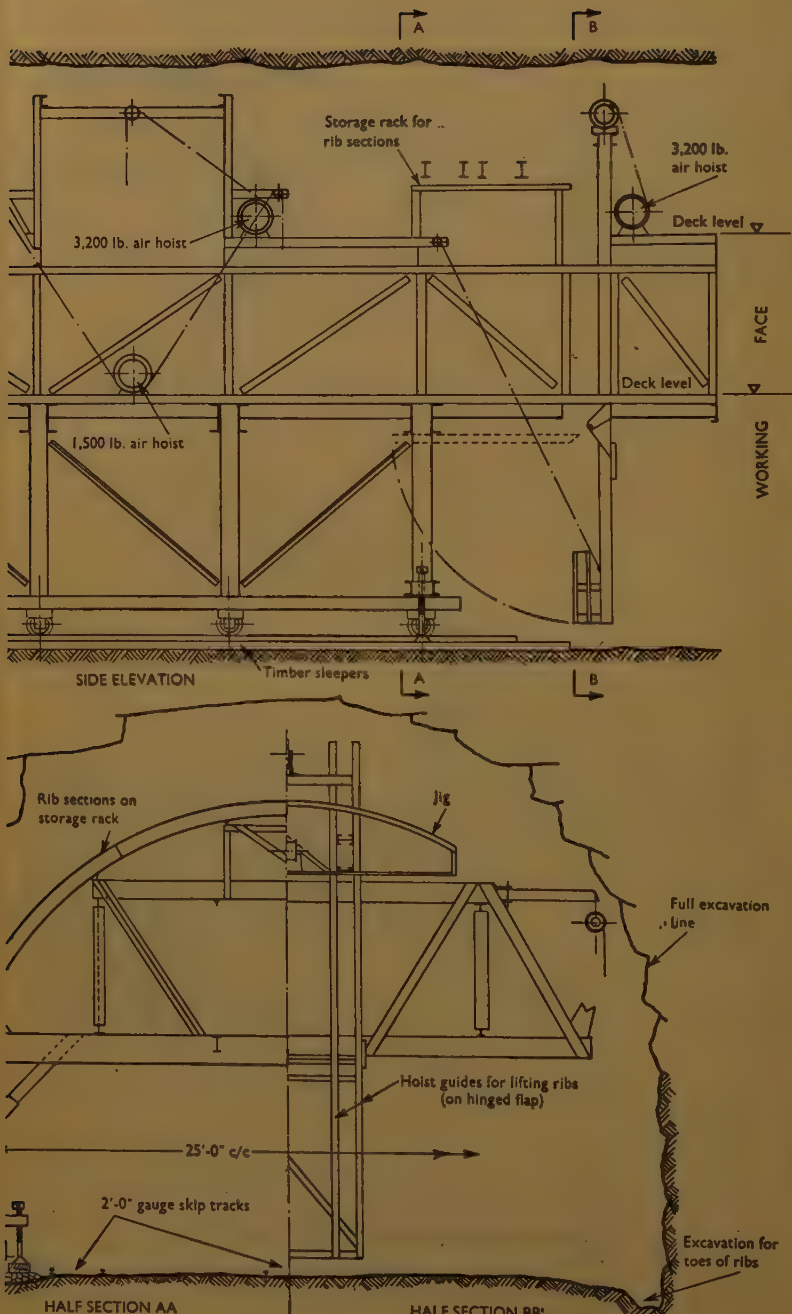
LAY-OUT AT DUNFORD BRIDGE

WILLIAM CLOWES & SONS, LIMITED: LONDON

The Institution of Civil Engineers. Proceedings, Part I, September 1954

P. A. SCOTT and J. I. CAMPBELL

Figs 14



SIDE ELEVATION

HALF SECTION AA

HALF SECTION BB'

William Clowes & Sons, Limited: London

ideas about the contract, the greatest co-operation had persisted throughout.

The people who had built the original tunnels a hundred years ago had been very fortunate in that economic considerations had forced them to drive a small tunnel. It was, however, a pity that more money had not been available at that time, since if a start had then been made with a bigger tunnel, the experience gained would have proved invaluable to the contractors engaged on the new tunnel. The bigger tunnel was probably the right answer from an economical operating point of view but, as Mr Scott had stated in his introductory remarks, had it been known that the shale was going to behave in the way that it did in the larger tunnel, then the planning and lay-out would have been very different. It was always easy to be wise after the event!

Probably the greatest innovation that had been introduced had been the by-pass tunnels. It had given the opportunity of opening up throughout the whole length of the large tunnel with break-ups—making it possible to erect ribs in probably a dozen places instead of two or three.

The lining had been quite a triumph! A big job had been anticipated in lining the 25-foot tunnel, but it had been found that there was no difficulty in getting the footage—in fact, progress had been almost twice as fast as had been reckoned in the original programme.

The main difficulty had been the shale, and Sir Andrew asked all those who read the Paper to pay great attention to shale, particularly that type of shale, because it looked so innocent and yet it was liable to break away suddenly without warning. It was fortunate that more lives had not been lost in the tunnel. The men had run considerable risks and the way in which they had made safe places where there had been collapses was most commendable.

Lt-Col G. R. S. Wilson said that he was pleased to have been associated with one of the greatest civil engineering works carried out on British Railways and one which had proved to be much more difficult than had been anticipated from a study of the old tunnel records.

The description of the behaviour of the shale formation, which was neither sound rock nor good plain earth, was especially valuable. By means of a concrete lining the tunnel roof had been supported far more permanently than could have been achieved by the use of masonry and brickwork. There was little fear that circumstances or deterioration would require the building of a third tunnel 100 years hence.

A study of the Paper, coupled with a very informative visit to the works, had given Lt-Col Wilson confidence for the recommendation that he would have to make later in the year for the Minister of Transport's statutory approval of the tunnel from the point of view of the safety of the traffic using it. That approval would have to be given after an inspection of the finished work which could only be of a very superficial nature, just as the old tunnel had been inspected by his predecessor, Captain George Wynne.

Lt-Col. Wilson then read two inspection reports which referred to the second of the old tunnels completed in 1852. The first report, dated 16 January, 1852, was as follows :—

"I have the honour to acquaint you, for the information of the Lords of the Committee of the Privy Council for Trade and Plantations, that I this day inspected the Woodhead Tunnel on the Manchester, Sheffield and Lincolnshire Railway. The tunnel, which is constructed for a single line of rails, is driven entirely through rock and is lined throughout with masonry [*sic*]. Near the centre of the tunnel and for a length of about 15 yards one of the side-walls has bulged. The contractor is at present engaged in taking it down and rebuilding the bulged portion, and until he has completed this work I do not consider that the tunnel will be in a fit state to be opened. I have therefore to report that, owing to the incompleteness of the masonry [*sic*] of the tunnel, I am of opinion that the line of railway which passes through it cannot be opened without danger to the public using the said tunnel."

On 21 January, 1852, Captain Wynne had submitted a second report as follows :—

"After my inspection of the Woodhead Tunnel on the Manchester, Sheffield and Lincolnshire Railway on the 16th instant, I had to report that, owing to some defects in the masonry [*sic*], it was not in a fit state to open for public traffic. As I was then on my way to Newcastle on public business, the engineer of the line begged that I would on my return re-inspect the tunnel, as he hoped by that time to have the objectionable parts of the masonry [*sic*] taken down and rebuilt. I accordingly examined the tunnel again on the 19th and found that the part of the side-wall which had bulged from the cause I have already explained in my former report, had been taken down and rebuilt, and I am now able to report, for the information of the Lords of the Committee of the Privy Council for Trade and Plantations, that in my opinion, the Woodhead Tunnel is in a fit state for the conveyance of passengers."

Thus it had remained. Captain Wynne had been a man of few words, but Lt-Col Wilson had little reason to fear that there would be any bulges when he came to inspect the tunnel, supported as it was by the monolithic concrete lining. The concrete had been placed under pumped pressure, supplemented by grouting, and through most of the tunnel it was of considerably greater section than the planned 21-inch arch, which in itself had been shown to have a high factor of safety even without the adventitious reinforcement of the embedded steel ribs.

Generally the Paper showed that, whilst modern equipment greatly assisted the economical construction of large civil engineering works, the unpredictable forces of nature gave rise to the same difficulties and dangers as in the past, and just the same courage, ingenuity, and resourcefulness were necessary to overcome them.

Had any provision been made for drawing away water accumulating

behind the concrete lining and had any special precautions been taken to prevent water from seeping through imperfections in the crown of the concrete arch and affecting the insulation of the 1,500-volt overhead line? Lt-Col Wilson noted that standard flat-bottomed track had been laid through the tunnel. Had welded joints been used, or had it been considered that that would make eventual re-railing in the tunnel too difficult? Presumably the rails would have a much longer life in the tunnel than would be the case under steam operation.

Mr J. C. Waddington said that the Woodhead tunnel was probably the first major railway tunnel to be constructed in Great Britain for over half a century, and it might be expected that, with the advances which had been made in equipment and materials, great progress would have been achieved over the methods used for constructing railway tunnels in the last century.

The present result was certainly spectacular. Large quantities of steel, approximately 6,000 tons or just over 1 ton per linear yard of tunnel—probably at a cost of nearly £500,000—had been used in the work for temporary support. Large quantities of overbreak, involving replacement with concrete, had occurred. Rock falls of almost unprecedented dimensions had developed. The concrete lining had been of massive construction, large supplies of modern plant had been used, and the whole project was generally of an impressive nature, resulting in an expenditure of £4,250,000, which for the length of the tunnel gave a cost of £900 per yard. That was approximately ten times the cost of a similar tunnel if constructed half a century ago, although the present wage rates were only about four to five times those prevailing at that time. Napoleon Bonaparte had stated, after a very gallant and costly attack on a position which had been secured at great cost in life and material, "It is magnificent, but it is not war." Mr Waddington felt the same about the Woodhead new tunnel—it was very spectacular, but hardly economic.

At the time of the preparation of the tenders for the work, both the engineers and the contractors had been in a very advantageous and almost unique position, since there was in existence (as mentioned on p. 508) a detailed longitudinal section describing the geological features encountered in the driving of the original Woodhead tunnels on the same line and level and approximately 100 feet away. It was not often that engineers had available such precise information on a tunnelling project. A study of that section had revealed that rather more than half the total length of the tunnel would be through shales and thin-bedded rocks, or with the arch of the tunnel mainly in shale. Shale was well known as a ground which was often difficult to hold, because of the pressures exerted as a consequence of swelling on exposure to air, and such ground was usually included, along with clays and marls, in the technique of soft-ground mining.

The contract drawings and documents had included only one type of lining—concrete. Within that limitation there had been only one method

of construction open to the contractor, namely, to open up the tunnel "full-face" in considerable lengths supported on temporary steel supports, and then to place the concrete in the volume required for the lining and the overbreak by means of concrete placers or pumps, using large travelling shutters of very great strength to hold the loads imposed by the depth and mass of wet concrete. The tenders had been submitted on that basis.

During the railway era in Great Britain, a system of tunnelling which had become known as the English system had been developed on the railways for use in soft-ground conditions. The system consisted essentially of the excavation to full-face of short lengths of tunnel (about 15 feet to 20 feet at a time), and supporting the newly-excavated roof by means of bars of timber or steel, carried at one end on the already constructed permanent work and at the leading end by props and sills secured on benches in the face. On the completion of excavation, the permanent work, including foundations, side-walls, and arch were built in brickwork turned over in one operation, thus ensuring that the ground opened had little opportunity to exert pressure by swelling and exposure to the air. The brickwork would normally be five or six rings in thickness and the voids resulting from overbreak could be readily packed with rock in mortar. As the brickwork progressed, most of the bars supporting the roof could be struck and withdrawn or built in, and the method of construction ensured that the ground opened out was adequately secured and held, and at the same time resulted in great economy of material. That system had become known as "length-by-length work" or "bars and brickwork" and, apart from railway tunnels, had also been used extensively for main sewer construction prior to the 1939–45 War.

The concrete lining design which had been adopted was suitable only for the ground conditions met in the self-supporting massive sandstones and grit-stones at the Woodhead end of the tunnel, and Mr Waddington submitted that great economy could have been achieved by using an alternative design in brickwork for the lengths of tunnel in shale demanding a soft-ground technique of mining. It might be argued that bricklayers would have been very difficult to secure. However, in building side-walls and arches with five, six, or more rings of brickwork, the only real skill required was in keeping the face of the tunnel true to line and level, and the placing of the mass of brickwork and rock filling in the voids behind involved little more than semi-skilled labour. Expensive aggregates had had to be brought in from outside sources for the concrete. Bricks could have been brought in just as easily from the Lancashire, South Yorkshire, or Midland areas, where good quality facing and backing bricks were available.

Tunnelling methods requiring the temporary support of long lengths of tunnel with concrete lining constructed in comparatively long sections had been developed mainly in the United States of America and were economic methods where hard igneous or self-supporting rocks occurred or where

large quantities of cheap timber or cheap steel were obtainable, as was the case in America. However, during the period of construction of the Woodhead Tunnel, there had been an extreme shortage of steel in the United Kingdom and licences had been necessary to obtain fractional quantities of a ton of steel, so that it hardly seemed logical to have entered upon the prodigal expenditure of 6,000 tons.

Surely the use of brickwork and steel bars, which could have been disused tram rails purchasable without licence and very cheaply, would have resulted in a considerable economy?

There was a great deal to be said in favour of a pilot tunnel running separately from, but parallel to, the main tunnel. Such a pilot tunnel would have made possible an attack on the main tunnel at many points, facilitated the use of "length-by-length work," and considerably advanced the rate of construction.

Mr Waddington agreed with Mr Scott regarding the time that had been allowed for the contract. There should have been at least 5 or 6 years available, which would have made the use of a soft-ground mining technique possible.

Mr R. E. Sadler said that the old Woodhead tunnel had been the first great railway tunnel to be built anywhere, and it had certainly been the one upon which many others had been based. The temptation now was to ask whether the Woodhead new tunnel would be the last of the great railway tunnels seeing that the cost of such works was now so high.

It was mentioned on p. 507 that the ratio of costs between two new single-line tunnels and a new double-line tunnel was 1·43 to 1·00. Since there had been difficulty in fixing any bonus rates for the large tunnel as opposed to the headings, perhaps that ratio might now be amended so as to be in favour of providing two single-line tunnels, which was, after all, only carrying a little farther the idea which had eventually been utilized on the site, in the employment over a considerable length of the tunnel of an auxiliary by-pass tunnel. When single-line tunnels were provided as a general rule, it was subsequently much more difficult to maintain the track, but perhaps that was a problem which could be mitigated with modern equipment. Certainly, single-line tunnels were more favourable to the operating departments.

Mention was made on p. 510 of the alteration of gradients. The gradient of 1 in 129 in the tunnel was not the limiting one on that route; coming up to the tunnel from the west side it was 1 in 117. If the gradients had not been altered as shown (see p. 509), the only other method that could have been tried of allowing two trains in the tunnel simultaneously, on the same track, would have been by the use of trap points. However, it would of course have been very dangerous to put those in the tunnel. By altering the level of the tunnel as shown it would be possible in the future, when it became necessary, to reduce the transit time between the two stations of Woodhead and Dunford by about 2 or 3 minutes.

With reference to the section of the tunnel (see p. 511), that did, in fact, conform to the text of the Ministry of Transport Requirements, namely, that there should be a clearance of 2 feet 4 inches above the widest stock to be used.

Reference was made on p. 513 to making the tip sites look attractive. At the present time public opinion was very sensitive on that subject and that appeared to be the case even on those bleak Yorkshire moors. The main tip sites covered about 60 acres and after the main contractor had bulldozed them down to a contour that was reasonably acceptable to the various bodies concerned, it would cost over £10,000 to make them green again—or at least £165 per acre. There was a temptation to ask whether it would not have been better to spend the money on some of the ugly dumps that were near built-up areas.

In conclusion, Mr Sadler referred to the point Mr Waddington had made about the use of brickwork. Where would Mr Waddington have obtained bricks in sufficient quantity, of a suitably uniform shape, and at a suitable price?

Mr A. W. T. Daniel said it was a matter providing food for deep thought that there was the case of two tunnels, both over 100 years old, on which the maintenance and repair charges had become so high that it had become an economic proposition to build a new tunnel. That appeared to be the correct interpretation of the position, but it would be interesting to know whether the impending electrification had had any influence on the decision, because there had been statements elsewhere to the effect that if it had been decided to repair the existing tunnel, there would have been difficulty in finding room for the overhead wiring. Mr Daniel would like to know whether, even if no electrification had been in prospect, the decision to build a new tunnel would have been taken, since in the near future many similar problems would arise elsewhere in Great Britain.

Three alternative schemes were mentioned on p. 507, and the ratio of costs of proposals (1) and (3) was given as 1·04 to 1·00; on that basis scheme (3) had been recommended. Thus an important decision had been based on a delicate balance of costs, and in view of the difficulties which had been encountered and the changes which had had to be made in the methods of excavation, it would be interesting to know whether the same ratio of costs still held good. It would probably be correct to say that one of the chief merits of the scheme adopted was that, once the tunnel was complete and the track connexions at either end made, the work was to all intents and purposes finished, whereas in scheme (1) the work would have had to be done in two stages. Furthermore, the programme could be so arranged that no steam trains traversed the new tunnel.

Reverting to the question of the age of the old tunnel, it would be instructive to know the redemption period which had formed the basis of the loan of 1837, because if that had been less than 100 years (as had probably been the case) then the tunnels could be said to have more than justified

their first cost, and the construction of a new tunnel or tunnels would be the correct policy financially.

It was stated on p. 510 that the gradients were arranged to rise to a summit within the length of the tunnel, instead of having a uniform gradient throughout, and that that had been done to facilitate the signalling that would be required should it be necessary to allow two trains on one line in the tunnel simultaneously. Presumably that was to allow for a breakaway with loose coupled wagons, since otherwise there appeared to be no advantage in steepening the gradient from 1 in 203 to 1 in 129 even with electrification.

On p. 537 a few notes were given on the survey, but it was hoped that the Authors could elaborate them, with the aid of diagrams, showing the method of measuring the base-line and the angles, how many times each was measured, and what degree of accuracy was obtained. It would also be helpful to know what instruments had been used. In view of the fact that the straight-through chainage of the old tunnels had never been easy on account of smoke, presumably that had now been done through the new tunnel, and perhaps the Authors would not object to stating the closing error.

If a through sight from one end of the new tunnel to the other had been taken after completion, could the Authors say whether any attempt had been made to check the levelling as well as the survey, taking into account the combined error due to curvature and refraction, which on a distance of 3 miles was over 5 feet ?

It would be interesting to know whether a future useful purpose had been found for the old tunnels, or was it considered necessary to fill them in, or render them safe in some other way ? That was an important item in the cost which presumably had been allowed for.

Mr J. D. Dempster said that, as the Resident Engineer on the Woodhead tunnel for $4\frac{1}{2}$ years from its commencement until the end of June 1953, it might be unwise of him to ask any awkward questions !

Mr Dempster supported the recommendation made by the Authors about the use on large repetitive works of the services of the time-study engineer. Whilst that type of information was normally taken to apply to factories, there seemed no reason why the civil engineer should be above accepting its usefulness. If it could produce reductions in cost and savings, meaning an increased profit to the contractor and a decrease in the cost of the job, surely it would give satisfaction to all concerned.

Reference was made in the Paper to the use of the imprest account target system, in view of the unusual construction problems involved. Obviously, that system was used only as a last resort where, for one reason or another, a contractor was unable or even unwilling to tender a price which was acceptable to the engineer and the employer. From Mr Dempster's experience at Woodhead and, strangely enough, experience since then in other places, that type of working was rather unsatisfactory.

However, the simple system used at present needed a great deal of improvement. It was necessary to adjust the incentive element and, at the same time, to allow control on established lines. The adjustment could be done by simulating the conditions of a priced contract—by providing equal incentives to avoid loss and to obtain profit. The reformation of the terms might at first sight seem to be rather involved, but it was no more so than the contractor's own method of safeguarding his interests on a priced bill.

*** **Mr William Cathrow** observed that excessive overbreak appeared to have been the main difficulty encountered during the driving of the tunnel. Overbreak was cumulative in its effect. It increased the unsupported surface area of the tunnel, thus leading to further overbreak. The cost of filling those large spaces with concrete added considerably to the cost of the contract.

The proximity of the new tunnel to the old ones (only 100 feet centre-to-centre) was an advantage, since water seepage into the old tunnel would have helped to drain the surrounding rock formation. However, the question arose whether the maximum thickness of only 70 feet of solid rock between the new tunnel and the old tunnels was sufficient to avoid the area of internal stress in the rock surrounding the old tunnels. Unbalanced stresses must exist in rock close to a tunnel. The concrete lining did not exert external pressure on the rock face, so that there was no external force to counteract the internal stresses set up in the rock by the driving of the tunnel.

Pressures under the earth's surface seemed much higher than would have been anticipated. That no doubt arose from the huge forces which in past geological times forced enormous masses of rock upwards. Internal stresses might also have been caused by geological cooling. The release of such forces in the rock surrounding the old tunnels might have so disturbed the shale formation that that was the main cause of the trouble with the new and much larger tunnel immediately alongside. Furthermore, during the course of years, drainage of water from those shales through the old tunnels might have resulted in shrinkage and, later, in the formation breaking up as a consequence of the high pressures and unbalanced stresses.

If the above deductions were correct, then the trouble could have been reduced by curving the new tunnel away from the old ones; so that, although it avoided the disturbed area of rock, it did not miss the area drained by the old tunnels.

Normally trouble with water was just as serious as overbreak. In the Woodhead new tunnel there seemed to have been no trouble at all with water. That was of course a most important point, since bare high-tension electric conductors were to be used in the tunnel. However, if records of

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*** This and the following contribution were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

the driving of the old tunnels were available, it would probably be found that the main difficulty then had been water and not overbreak.

It might be said that since the second single-line tunnel had been driven without special difficulty, a few years after the first, there should have been no trouble with the third tunnel. The second tunnel, however, had been driven before earth tremors, vibrations caused by trains, seepage of water through the rock, etc., had altered matters. Furthermore, both the first tunnels were much smaller than the new one and a century ago the explosives used were probably not so violent.

Mr A. M. M. Wood observed that the relative costs of the three alternative schemes considered (see p. 507) were most surprising, especially in view of the hazards involved in driving a double-line tunnel in that locality. Perhaps the Authors in the light of experience gained since 1946 would state how the ratio of costs should now appear.

It seemed unlikely that the two old tunnels would be maintained in their present state of repair, in view of the cost. However, if the tunnels were allowed to fall from disuse into ruin, would any anxiety be felt because of the possible effects of their collapse on the new tunnel ?

A geological section of the new tunnel was not provided, so it was impossible to decide where any such effect might occur. However, from a study of the relative gradients, the crown of the new tunnel at its summit was about 50 feet above rail level in the old tunnels alongside ; but since stratigraphical considerations limited " draw " at that point, there was no cause for anxiety. Was any consideration given to depositing a proportion of the spoil on a temporary tip, so that it could have been used later to pack the disused tunnels ? That would have entailed double handling and anyway might have been contrary to the requirements of a vigilant planning authority.

The tunnel arch analysis described on p. 511 (see *Fig.* 4) seemed rather odd. The lining thickness of 21 inches might well have been established by consideration of stresses in the concrete for a maximum postulated loading. Instead, it was related to a requirement to maintain the thrust line in the middle-third, although such a result depended upon quite arbitrary (but reasonable in magnitude if the necessary grouting was achieved) passive forces developed by the surrounding rock. Another point associated with the calculation of rock load appeared rather distorted. It occurred on p. 523 in the reference to rib-support strength. The maximum stress in an arch rib in a tunnel of known diameter and rock load was a sum of two components ; the first was a constant and the second was proportional to the square of the distance between the blocking points. The figure of 55 per cent was thus seen to relate to a single instance and had no universal significance, as the context appeared to suggest. According to *Fig. 13* the temporary supports were expected to withstand the full roof load for periods as long as 18 months. The decision, taken in the design stage, to consider the supports separately from the finished lining, appeared rather

premature and as a consequence no allowance had been made for the strength of the steelwork built into the lining. It would appear that the basic defect of the supports was the haphazard arrangement of the steel roof-packings (shown by *Fig. 25*).

Would not more efficient use of the concreting plant have been obtained if the travelling shutters had been split into shorter lengths ? Two or three shutters at each end of the tunnel, each 20 to 30 feet long, would have permitted a much smoother rate of concreting.

Mr Campbell, in reply, agreed with Sir Andrew MacTaggart that it was very easy to be wise after the event. That had been very much the case at Woodhead—not that he wished to imply that any serious mistakes had been made.

The question of a single-line tunnel or a double-line tunnel was very problematical and could be argued indefinitely—and perhaps the only way to resolve the matter would be to construct one of each kind, which would be a rather expensive proposition !

With regard to the contract document, it was impossible on such a job, with unpredictable risks, to be orthodox. Equally, the prime-cost contract in its normal form was welcome neither to the contractor nor to the employer, and he considered that the method adopted had been a half-way house. It was not, however, the solution, and some other factors needed to be introduced to make it entirely complete.

To Lt-Col Wilson he would say that he was not responsible for the track, but (answering on behalf of his colleague on the London Midland Region) he would say that consideration had certainly been given to welding the rails but time had been the ruling factor. At present, he thought, they were butt-jointed, but he was sure that serious consideration would be given to welding the rails on the next re-railing.

Mr Daniel had suggested that the high maintenance charges on the old tunnels had been the reason for the new tunnel. It had not been that the charges were high, but the extent of the repair work and the necessity for absolute possession, which it had been quite impossible to obtain with the volume of traffic passing through the tunnels. Mr Daniel had also mentioned the factors of 1·04 and 1·00. Had a single-line tunnel been driven, followed by the repair of one of the existing tunnels, it would have been necessary to have possession of that centre tunnel after the new single-line tunnel had been constructed. There again, the time factor would have come in, because it would have been quite a long job to reline the existing tunnel by reason of the fact that the rock would have had to be trimmed. There had been a scheme before the 1939–45 war for carrying the overhead electric equipment through the existing tunnel, but the clearance for the traction wires had been much less than that desirable ; nevertheless, it could have been done. Mr Daniel had referred also to the basis of the 1837 finance. There was no trace of what that basis had been. With regard to the gradients, by having a rising gradient, followed by a

falling gradient, it would, by the introduction of suitable signalling safeguards, be practicable to have two trains in the tunnel on the same line at one time and thus increase line capacity.

Mr Scott, in reply to Mr Ratter's question about the pilot tunnel, said that that was a matter on which the contractors could give a better answer. Undoubtedly the level of the pilot tunnel as chosen for the radial drilling system would not have been adopted had it been decided to drive a pilot tunnel and enlarge it to full section by face drilling. The floor of the pilot tunnel would most certainly have been at the invert of the enlarged tunnel. As to whether, had the Authors known all that they now knew, they would have driven a pilot tunnel at all, he thought that the contractors would probably agree with him when he said that, if they were to start again now, quite probably they would drive the access tunnel to one side of the final tunnel and would not drive a pilot tunnel at all; they would merely drive a long adit and go in at various points to drive the full-section tunnel straight away. That would lead to ventilation difficulties but he could think of no others, and undoubtedly the ventilation problem could be solved.

Mr Ratter had also asked about the method of transporting the spoil from the tunnel to the tip sites, and that, again, was very largely a question for an experienced contractor; but he thought that most people would agree that the system which had been adopted was as good and as economic as could be achieved today and that the conveyor-belt system, whilst it might have shown an approximately similar price, would have been unlikely to show any great saving.

In reply to Lt-Col Wilson, the provision for the drainage of water behind the concrete lining was referred to on p. 535, but perhaps the wording as given was not quite clear. The method there described had been applied throughout the tunnel; that was to say, the policy had been not to block the flow but to lead the flow round in channels eventually to the centre drain provided in the tunnel itself, and so to prevent the building-up of high pressures behind the tunnel. Drips and trickles of water, varying with the recent rainfall, had come through the contraction cracks in some places, and two steady flows occurred where contraction had opened construction joints. Those leaks were diverted down the walls by means of bitumen strips fixed to the surface of the concrete. The leaks had a small nuisance value but constituted a very small fraction of the total flow dealt with by the drainage system.

Mr Waddington's remarks deserved considerable thought. The cost of steel was a little terrifying at first, but when looked at in the broad picture of the whole tunnel, Mr Scott did not think that the cost of the total tonnage of steel was out of the way, nor did he think that the country had suffered as a result of the 6,000 tons of steel which had been put into the tunnel. A large number of the bank bars had been of the tramline variety.

With regard to the available very good geological section of the tunnel 100 feet away the Authors had hidden nothing. Naturally they had thought that they would meet somewhat similar conditions. The conditions had indeed been somewhat similar, but they had also been a great deal worse than had been anticipated. Borings, of course, would have told no more. To get borings would have meant a 500-foot hole in each case, and what would have been pulled out would have told very little more than was known from the geological section. As an illustration of the unreliability of that advance information, he would only cite the experience of the contractors at the Dunford end when starting to drive the access tunnels only 50 feet south of the pilot tunnel. Of course, they had known exactly what there was in the pilot tunnel, having just gone through it, but in one notorious section the shale encountered had indeed been—as Mr Waddington had said—something that had to be treated as soft ground—in fact, it had been so soft that it was nearly soft earth. That in itself gave part of the answer to the question regarding the twin tunnel. Had two tunnels been driven instead of one, the second tunnel would have run through that very poor section of rock, and one did not know what other difficulties would have been found had a second tunnel been driven parallel to the one already driven.

Mr Waddington had then referred to the cost of the lining. Mr Scott was glad that Lt-Col Wilson had given approval of the very solid lining that had been provided. Mr Waddington, however, had asked why brick should not have been used. Brick had been considered and so had the English system. One inevitably thought of the English system when meeting such poor conditions. A year or two before the Woodhead tunnel had been driven, there had been occasion to drive another tunnel not far away, and there again, although it had been a single-line tunnel, it had been found that there was very similar rock formation and it had been necessary to support the rock with steel ribs for a very large portion of the length. In one section, somewhat in despair, they had put in brick, and the cost had been so appalling that they had had to go back to concrete, with all its overbreak and all its attendant difficulty of importing an expensive aggregate. That was the nearest to an answer that he could give to Mr Waddington's question. In that matter, he was glad that Mr Sadler had asked the question as to where the necessary number of bricks could have been obtained. Mr Waddington had suggested that had there been 5 or 6 years available time for carrying out the contract, perhaps the contractor might have considered brick; but the fact was that the railway programme had not permitted anything like 5 or 6 years, and the programming had had to be done accordingly.

Mr Scott did not quite understand the point that Mr Sadler had made about the bonusing of the job had two tunnels been driven instead of one; but again, that raised the question of the unknown type of rock through which the second tunnel would have gone, with which Mr Campbell had

dealt. He was glad to have had Mr Sadler's agreement with regard to the beautification of the tip sites. It was a difficult problem today, with the Town and Country Planning Act, to know just how far to go in such beautification, but undoubtedly one had to draw the line somewhere when costs rose as they did in providing the near-ideal which lovers of the countryside asked for.

Mr Daniel had referred to the relative costs as between systems Nos 1 and 3. The decision had indeed been made on a delicate balance of cost, but it had not been entirely because the cost had been 1 as against 1·04. There had been a preference for the single double-line tunnel as against the two single-line tunnels, and since it had also happened to appear to be cheaper, it had been chosen. Mr Daniel had asked for details of the survey, and Mr Scott stated that only those aspects of the survey work which had seemed at all unusual had been mentioned in the Paper, since the triangulation, the underground traverses, and the shaft plumbing, had all been carried out using well-established techniques which had been described very fully in various Papers on tunnelling in the past 20 years. Mr Daniel had asked what accuracy had been obtained, and what instruments had been used. Generally, angles had been measured to ± 1 second of arc with 5-second micrometer theodolites reading directly to 10 seconds arc, and between two and six sets of observations (each of 8 turns) had been needed to establish the angles. Observations had been arranged so that any error in the chainage of the base lines had had a negligible effect on alignment, but in spite of the difficulty of measuring through the old tunnel in very limited periods, the two values obtained for that length had differed by only 0·145 feet in 15,904 feet. No through sight in the new tunnel had been possible, because of the change in grade and the curve at the western end; all levels had been determined by the normal method, both above and below ground.

Mr Cathrow had spoken of the effect of the old tunnels on drainage and rock stresses in the area of the new drive. The pattern of ground-water levels and ground-water flows in the fissured, jointed, and faulted layers of shale and sandstone was exceedingly complex. Although on one occasion a temporary flow of about 800 gallons per minute had been tapped in the new tunnel, disappearing through a fissure and greatly augmenting the flow in the old tunnel drains (to the consternation of the permanent way gangs), the general picture of water inflow before lining was that the new tunnel seemed to attract quite its fair share of ground-water compared with the old tunnels, and it seemed doubtful whether the two tunnels affected each other as much as might have been expected. As for disturbance of the rock stress patterns, all experience with that particular type of rock indicated that the redistribution of stress was generally very local, except in poor rock which had been allowed initial movement in a zone above the excavation, spreading out at perhaps 30 degrees from the vertical. The new works were of course well clear of such a zone. The

overbreak could be accounted for generally simply by the mechanical weakness of the rock in bulk and the progressive movement of the layers when disturbed, assisted by the softening of the shale on exposure and on the alteration of the ground-water flow-patterns.

The local nature of yielding in the strata penetrated was also the answer to Mr Wood's query about the effect of collapse of the old tunnels on the stability of the new one. No effect was expected, but in any case the new tunnel lining had a considerable factor of safety against any likely stress from such a cause. Backfilling the old tunnels was not considered necessary; the by-pass passages had been fully backfilled to 20 feet from the nearest point on the tunnel lining, but beyond that the unsupported roof spans left had been limited in relation to the type of rock and the distance from the tunnel.

Regarding Mr Wood's remarks on the tunnel lining, it was considered that the real risk of failure in the lining arch was from tension developing as a result of failure to develop the necessary passive pressures; the middle-third analysis was quite illuminating in illustrating the need for those pressures and their magnitude. The compressive strength of the concrete was adequate for enormous loadings. However, it was not claimed that any analysis was more than a guide in arriving at the design. The figure "of up to 55 per cent" for the reduction in stress in a rib well packed to the rock, compared with the same rib packed in few places, was computed as an illustration of the magnitude of the effect; it depended on an estimate of the minimum packing anyone was likely to put in, and whilst it was agreed that the figure had no special universality, it was worth while being specific on the reduction. He could not agree with Mr Wood, that the decision not to allow for the strength of the steelwork when designing the lining had been premature. The nature and amount of temporary supports erected could not be decided before excavation had actually taken place—any attempt to do so would have resulted in great waste. Nor could the lining thickness be adjusted from length to length. It was agreed that the roof packings were far from ideal, but it should be remembered that they were thrust in hurriedly by men working as best they could under unsupported rock of a very unreliable nature, and often it was a case of getting something in quickly at all costs. He had found from his experience of supervising the contract from his London office that when criticism was attempted from the top stage of the gantry at the face, rather than from safer situations, it was apt to undergo modification! The use of shorter shutter lengths would have very seriously slowed down the possible rate of lining, as well as requiring additional costly stunt ends.

Correspondence

Mr E. W. Cuthbert observed that the decision to disregard the published desirable structure gauge of the Ministry of Transport was difficult to understand for two important reasons. First, the tunnel

was on a line carrying very heavy traffic, and, secondly, the tunnel was of exceptional length. The profile adopted not only infringed the desirable gauge by 6 inches, but at points 11 feet above rail level was 2 inches less than the minimum gauge. It must, therefore, be concluded that the published structure gauges of the Ministry of Transport were unnecessarily large and that they should be amended to accord with reasonable practice.

One mile of the tunnel was to a gradient flatter than 1 in 1,000. Such a gradient could lead to difficulties in dealing with seepage water, and it would be interesting to learn how satisfactory drainage had been provided in the flat section.

The Authors had described how the shale softened when wet. Mr Cuthbert wondered, therefore, what the final condition of the formation would be, particularly where the gradient was 1 in 1,186. There, any slight irregularities in the trimming of the shale could lead to seepage water collecting in pools. Was there any risk of such water percolating into fissures opened during excavation operations and, in course of time, softening the shale to such an extent that it could no longer strut the haunches apart ? Would not the provision of a concrete invert or concrete struts have been less expensive and of more permanent value than the heavy haunches actually constructed ?

Mr Duncan Kennedy considered that the description of the difficulties encountered in the driving and lining of the tunnel and of the means used to overcome those difficulties formed a useful addition to the published records of such works. The subject of tunnel construction was much in the minds of civil engineers at the present time because of the large number of tunnels being constructed in various parts of the world, chiefly for hydro-electric schemes. It was interesting to note from the Paper that, because of the difficulty in placing the concrete lining in horizontal layers at a sufficient speed, a sloping-flow method had been adopted, and that pneumatic placers would have been tried if they had been obtainable.

The Authors had not described the quality of the finished work, and their views on that would be interesting. The flow conditions would no doubt have been mitigated by the retarding influence of the ribs where the latter had been used, and by the fact that the concrete had been pumped in at from two to five points.

It had long been an axiom with engineers that concrete should not be allowed to flow in the work, but should be deposited as nearly as practicable in its permanent position, because of the tendency in flowing concrete to segregation of its various constituents. Whilst it was never wise for an engineer to follow, without question, any particular practice merely because of its general acceptance, yet in the case of flowing concrete there was ample evidence in support of the objections to its use. Where that method had been used in new work the varying quality of the layers could usually be detected without much difficulty, and in older work which had been subject to weathering, sea or river action, or other

destructive influences, the lines of flow were obvious where the weaker layers had disintegrated.

It should be recognized, however, that durability was not the only factor that had to be considered. Cost was important, and it sometimes happened that quality of workmanship had to be sacrificed to speed of construction, because the earlier return on the capital invested offset the shorter life of the work. In the case of the Woodhead tunnel, conditions were favourable to the life of the concrete, which was not exposed to weathering nor to attack by locomotive smoke or other destructive agents, and which was easy to inspect at any time for signs of deterioration. For those reasons the same high quality of concrete was not called for, as was necessary in, say, an aqueduct tunnel, exposed to both acid attack and erosion, and which might have to function for years without an opportunity of inspection.

Pneumatic concrete placers had frequently been used in recent years for tunnel lining with qualified success, the better results being obtained in smaller tunnels, where the flow slope was shorter. Where criticism of the resulting concrete had been expressed, it had usually been on the grounds of honeycombing. That was of course evidence of segregation, but at least it had the merit that it could be seen, and to some extent cut out and rectified. A more insidious form of segregation was found at the other end of the scale, where layers of fines collected. The face looked good, but the softer parts of the concrete could be detected by close inspection. The tendency to segregation was increased with the wetness of the concrete, and in the worst conditions pockets or layers of laitance were formed, which had a consistency similar to clay.

Mr Kennedy had never seen any evidence produced to show that the placing of concrete by any type of machine did in fact remove the objectionable features of the flow method. If such evidence was not forthcoming, and there was no assurance that concrete of a uniformly high standard would be obtained where it was vitally necessary, then there was little merit in employing bigger and better machines to do the wrong thing. It should be recognized, however, that pneumatic placers had great possibilities because of their speed and simplicity in operation, and in awkward positions such as tunnel roofs they had advantages over hand placing. If the risk of segregation could be removed their usefulness would be much enhanced.

Mr Kennedy mentioned two tunnels that he had recently inspected where the concrete lining had been placed partly by hand and partly by pneumatic machine. By visual inspection and percussion he had found little difference in the concrete placed by the two methods. In both cases the concrete was mixed outside the tunnels, transported from 1 to 2 miles by mixer-truck, and on arrival was mixed again for a minute before being discharged into the pneumatic placer. The time between initial mixing and placing in the forms varied from $\frac{1}{2}$ to 1 hour.

It was a matter of experience that concrete, a short time after mixing, began to lose some of its fluidity and became more plastic and cohesive, and therefore less liable to segregate. The same applied to mortar, and that was the reason the bricklayer, when building with cement mortar, liked to let it partially set for a time before using, since the bricks were then not so liable to slither about on the wall whilst being laid.

There was a long-standing belief that when concrete had passed the initial set it would lose strength if disturbed again. That was a mistake. Tests showed that if freshly made concrete was kept for 2 or 3 hours or even longer and then turned over again it would, if still workable, have actually gained in strength so long as no additional water was mixed with it. There was food for thought there in dealing with the problem of segregation, and the systematic delay in placing concrete where that was feasible might go a long way towards solving the difficulty.

It had been stated that for use in marine structures only one quality of concrete was suitable—the best. The same might be said of concrete which was liable to attack by acid water, such as was carried in many tunnels, and which was rarely available for inspection and repair. To ensure that concrete in such a position was really fit for its purpose two methods of test could be suggested. One method was to leave the work for 20 years and then go back and inspect it. However, that took time, in the course of which much damage might have been caused. The other was to cut a series of cores from the periphery of the tunnel at several places and test them. The knowledge gained would well repay the cost.

Mr A. W. Shilston stated that he was particularly interested in the target form of contract. In recent years, he believed that form of contract had been more frequently adopted than in the past. He felt that the use of the target contract in certain classes of civil engineering works might well have been influenced by the introduction in 1950 of the Standard Conditions of Contract, prepared jointly by the Institution of Civil Engineers, the Association of Consulting Engineers, and the Federation of Civil Engineering Contractors. The day appeared to be passing when contract documents would be laden with every conceivable type of obligation which had to be covered by the contractor in his prices. The assessment of such risks called for gambling, not skilled estimating.

Under any system of contract, provided the work was carried out efficiently, a contractor should be reimbursed his costs plus a reasonable profit margin. If the proposition that speculation was undesirable in the civil engineering industry was accepted—Mr Shilston believed that responsible contracting firms took that attitude—and the contractor had inadequate data on which to base an accurate estimate of costs (a state of affairs that usually prevailed in rock-tunnel work), his rates would either have to be inflated as an insurance against possible eventualities or he would have to speculate ; the former course was not always in the interests of the client and the latter was to be discouraged. The client, in addition

to having to shoulder the burden of "covering rates," might also have to meet further additional costs payable under clause 12 of the Standard Conditions of Contract which the engineer might authorize, or an arbitrator award, as the result of adverse physical conditions encountered. Thus the client might have to pay very much more for the work than it was worth. On the above reasoning, a case could clearly be made for departing from the conventional bill of quantities contract and recommending one based on the cost-reimbursement principle, provided adequate safeguards existed.

The Authors had reaffirmed the soundness of the decision to depart from the orthodox type of contract, but had nevertheless commented that the target contract in the form adopted was not entirely satisfactory. Could they indicate in what directions improvements could be made, even if perfection was unattainable?

Since the detailed method of operation of the target form of contract was not widely known, Mr Shilston asked if information could be given to indicate on what basis the tenderers were required to prepare their target estimates and how in practice the target figure had been revised during the currency of the contract; labour and material price fluctuations, of course, presented no great problem. For example, had the whole of the plant used been a Prime Cost under the contract; and had the bonus been a predetermined percentage of the savings, or stipulated by the contractor and included in his tender? Mr Shilston also enquired what criteria had been employed by the engineers in selecting the successful tenderer, assuming that all the contractors invited to tender were of comparable standing; also, in examining the tenders, had the price build-ups been available?

The Authors had inferred that greater responsibilities than usual had been shouldered by the Resident Engineer, in that he had to approve all the contractor's purchases and methods of working. It appeared that in regard to the control of constructional procedure, the Resident Engineer had no greater authority than in the normal bill of quantities contract; if a particular method of procedure was insisted upon which did not coincide with the contractor's ideas, the latter might claim (in both types of contract) that his profit (bonus) margin was being compromised. Would the Authors confirm that? Mr Shilston asked that question because he had always understood that relative freedom from claims and wider control by the engineer over constructional methods were some of the main advantages of that type of cost-reimbursement contract. Perhaps the answer was that if no bonus was at stake freedom would probably exist although, in other respects, absence of an incentive would be unsatisfactory.

Mr Shilston asked if an approximate percentage analysis of the final contract cost could be given under, say, four or five headings, together with the numbers employed on the Resident Engineer's staff.

In regard to tunnelling details, could typical drilling-round diagrams be given? Also, what minimum period had elapsed after concrete placing before grouting behind the tunnel lining had been permitted and what grout mix proportions had been used?

In conclusion, Mr Shilston enquired whether the consulting engineer would generally recommend future clients considering embarking on rock-tunnelling works to adopt a target in preference to the conventional form of contract.

The Authors, in reply, thought that Mr Cuthbert, in quoting the M.O.T. desirable standard structure gauge, might not have appreciated that under certain circumstances the Ministry accepted reductions on that profile to a minimum of 2 feet 4 inches over the maximum width over rolling-stock bodywork. The tunnel profile, as provided, was in excess of the latter dimension, although it impinged slightly at the cornice on the desirable structure gauge.

That modification had been adopted on economic grounds, for in such a major project, with a length of more than 3 miles, compliance with the desirable gauge would have very appreciably increased the costs. That slight infringement did not in any way detract from the effective clearances, since they were mainly related to the safety of men working on the line, and also, served as a safeguard in the case of passengers leaning from windows.

Mr Cuthbert had questioned the effect of water on the shale invert. As could be seen from *Fig. 3*, p. 510, there was a good cross-fall on the rock surfaces towards the centre drain; the actual gradient in that direction was 1 in 30, and there had been provision in the bill of quantities for the backfilling of overbreak to prevent the formation of pools where necessary. Actually, the concrete pad for the shutter track (seen in *Fig. 20*) covered the rock for a distance of more than 3 feet towards the centre from the footings, and after examination of the effect of exposure and traffic on the remaining strip of invert, wherever the quality of the rock was doubtful, it had been judged to be safe to permit all back-filling to consist of the 1½-inch limestone bottom ballast. The centre drain was generously proportioned, and allowed all the inflow to escape without the water level in the drain rising to that of the inlet holes, in spite of the rather flat gradient at the eastern end. The massive footings were considered more economical than a lighter structure with a concrete invert or struts.

Mr Duncan Kennedy's observations on the concrete lining were interesting and valuable. The sloping-flow method of placing had been adopted reluctantly in the first place, for the dangers of segregation had been much in mind, and the operation had been watched suspiciously. Inspection during placing, before setting, by examination of the surface finish, and of a few samples and patches cut from the walls and arch, had led to the conclusion that segregation took place only at high workabilities, and if, after a considerable hold-up, pumping was resumed without proper

precautions. There were a few places where flow marks on the surface had given evidence of some lack of uniformity, but at such places no areas of softness, such as were associated with serious separation of the fines, had been found. Honeycombing had been confined to a very few cases where there had been insufficient compaction around the arrises of recesses; such regions had been cut back to sound concrete, and the depth of honeycombing had never been more than 2 or 3 inches.

There had not been any apparent justification for going to the expense of cutting and testing a large number of cores from the lining, interesting though the results would have been, and consequently figures for the degree of uniformity and compaction achieved could not be quoted. It was the Authors' opinion, however, that the lining as constructed was as uniform and monolithic a structure as could have been achieved by any alternative method of placing, and that durability had not in fact been sacrificed to secure the speed and economy achieved in placing. The ample evidence of objections to the flow method of placing concrete, to which Mr Kennedy had referred, could be linked with the high workabilities often thought essential if pumping were used, and the freedom from segregation troubles at Woodhead might well be due to the fact that the average slump had been kept down to 1½ inch. Further information on the concrete workability and quality is contained in the Paper by Mr Sharman.*

Mr Kennedy's suggestion of deliberate delay between mixing and placing of concrete, to reduce segregation by allowing a partial setting to take place, might be rather dangerous where flow methods were used, unless very close control were maintained. It was not that there was any loss of strength when partially set concrete was reworked, but the possibility of completely set crusts and nodules moving about and interfering with penetration and compaction had to be considered.

In reply to Mr Shilston, the target cost had been arrived at in exactly the same way as a tendered price for a normal type of contract, and adjustments had been made to the target during the course of the works exactly as if the work were being measured for payment on the priced bill of quantities. All plant therefore had had to be included in the priced estimate. In considering tenders on a target basis, the contractors' proposals about fee, bonus, and penalty had had to be taken into consideration as well as the target itself.

Mr Shilston had asked why the Resident Engineer should have had any greater responsibility than would have been the case with a normal contract. If the incentive for the contractor, and the financial effect on the client, had remained unchanged by the adoption of a target system, there could theoretically be no difference, but the whole point of the system was to protect the contractor against loss if things went badly. That protection could only be provided at the client's expense, and with

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* See p. 564.

a limited penalty there was always the possibility that at some stage it might be found that the contractor was spending the client's money without himself being further affected. Even the possibility of such a situation imposed a duty on the consulting engineers to form judgements from the start on every plan of action and every item of expenditure, to an extent that would be quite unnecessary in the normal form of contract. The certification by the Resident Engineer of every supplier's account and every payroll increased the staff needed by the consulting engineers on site by about 25 per cent. That was quite apart from the extra effort required on the technical side, in order to maintain an independent and detailed knowledge of all problems encountered and methods used; the employment of a time-and-motion-study expert on the Resident Engineer's staff was an example of such effort.

There had been nothing novel about the drilling and charging patterns; a burn cut had been used for the pilot tunnel, and the spacing of holes and loading had been varied a good deal according to the nature of the rock. Grouting had not been carried out sooner than a fortnight after concreting, but there had been no necessity to reduce this period and it had not been regarded as the limit. Grout proportions had been varied over the whole range of consistencies in order to achieve penetration in the useful zone only. In reply to Mr Shilston's final query, as to whether the Authors would recommend the target form of contract for rock-tunnelling generally, it was felt that the determining factor was the extent of the risk lying outside the contractor's control. In any type of work, if that risk was thought by the tendering firms to be so great as to require excessive allowances in their estimates, the client might be well advised to adopt the target system, with appropriate provisions for maintaining adequate incentives over the whole range of expenditures considered possible.

Paper No. 6001

"Concrete Quality Control at Woodhead New Tunnel"

by

Frederick Andrew Sharman, B.Sc.(Eng.), A.M.I.C.E.

(Ordered by the Council to be published with written discussion) †

SYNOPSIS

The control of concrete quality has been dealt with in recent publications, both in general terms, and in detail for certain jobs where control was good. It has been shown that there is a striking difference, when judged by the variations in strength of works cubes, between the best practice in concrete production and the average efficiency achieved on works.

In the production of the 140,000 cubic yards of concrete used in lining the new 3-mile double-track railway tunnel between Manchester and Sheffield, plant lay-out was dictated by tunnel conditions and control methods could not be made ideal. At the same time, maintenance of a minimum quality was essential. A study had to be made of the tolerances, sampling and testing techniques, and economics applicable to large numbers of small batches, with a relatively high between-batch variation. Some of the considerations which emerged from this study, and from the records kept, may be applicable to the numerous cases in which control methods have to make the best of difficult conditions.

The Paper discusses the many factors which influenced the mix design for the tunnel lining at Woodhead, and describes the batching, transporting, and placing arrangements adopted. The question of relating samples to material in the work is discussed, and the sampling and testing scheme adopted is described. Results obtained are set out and analysed, and the value of statistical methods is emphasized. General conclusions suggested emphasize the importance of elasticity in specifying mixes, the necessity of co-operation between engineer, contractor, plant manufacturer, and materials supplier, the value of automatic plant, and the desirability of the wider dissemination of information about concrete quality.

INTRODUCTION

THE construction of the new 3-mile railway tunnel at Woodhead has been recently described.[1] An important feature of the work was the lining of the whole length with an in-situ mass-concrete arch, containing 77,600 cubic yards within the payment line. In addition, overbreak in excavation, amounting to about 67,000 cubic yards, had to be filled with concrete,

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† Correspondence on this Paper should be received at the Institution by the 15th January, 1955, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

[1] The references are given on p. 578.

and it was not generally possible to separate the overbreak filling from the concrete of the lining itself.

The dominant factor in deciding the programme and plant lay-out for the lining operation was the need to allow for a high rate of advance in concreting, without causing interference with the simultaneous driving of the tunnel faces. In this respect the scheme adopted was very successful: after the initial period, the average progress was 370 feet lined per week, representing an output of approximately 3,300 cubic yards per week; there was little interference with traffic to the faces.

From the point of view of concrete quality control, however, the system used had some serious drawbacks; it appeared, at first, that the margin beyond the specified minimum used in fixing the average strength of the designed mix was insufficient for the high between-batch variation observed. This led to a fairly detailed investigation of the relation between mix design, testing procedure, and the levels of quality control, as applied to a case where variation was inevitably high, and the required strength comparatively low. It was found possible to retain the original mix, and some conclusions of general application were formed during the work.

Design of the Mix

General Considerations

It is only possible to design concrete mixes at the specification or early planning stages of jobs if the placing conditions and available materials are foreseen in detail. The workability necessary for placing and compacting the lining concrete at Woodhead depended on the means adopted for getting the concrete into the arch and sidewalls, the nature of the supports and packings holding the roof during placing, the type of shuttering, and the speed of production. Until field trials had been carried out, using the actual placing plant selected and aggregates typical of those available, and simulating conditions behind the shutter, it was not possible to know what consistency was most suitable, or what range of workabilities could be dealt with by the pumps and compacting devices.

Once the necessary information on these points had been obtained, it was possible to work out details of the most economical mix. Instead of more or less arbitrary proportions being imposed on the contractor, the whole question was regarded as part of the construction method, allowing some flexibility in detail provided that the finished work conformed to design requirements.

Because of the nature of the contract at Woodhead, the Resident Engineer had a direct responsibility for economy as well as quality. In discussions about quality, therefore, he was obliged to think always of using materials to the best advantage, instead of leaving all considerations of cheapness to be urged by the contractor. The contractor, on the other hand, though concerned with economy, would have been heavily penalized

if any work had been condemned as a result of quality tests. Accordingly, a higher standard of control and a smaller margin between average strength and the specified minimum were asked for than the contractor at one time thought reasonable. Agreement was reached, but it is of interest to note that no generally accepted standards of control in concrete production, and few records of variation in the strength range concerned, could be found to guide these discussions. The original specification had, of course, been framed to guard against excessive economy of any kind and had only a limited application to these problems.

Before details of the mix can be discussed, it is necessary to describe the concrete-producing and placing arrangements so that their influence on the design can be appreciated.

Batching, Transporting, and Mixing

Aggregates in two sizes only ($\frac{3}{4}$ inch to $\frac{3}{16}$ inch and $\frac{3}{16}$ inch down) were led from conical stockpiles *via* conveyor belts in recovery tunnels to weigh-batch plants, one at each end of the tunnel, and each containing two weigh-hoppers. Cement was delivered in paper bags to sheds of 800-ton capacity. At the sheds the bags were split as required and the cement was fed in bulk to a cylindrical reservoir in the centre of the batcher, whence it was released into either of the weigh-hoppers. The general appearance of the batchers and feeds at the eastern end is shown in *Fig. 1*.

Each batch was made up by running coarse aggregate, cement, and sand in turn into the weigh-hopper to predetermined cumulative weights, and then discharging the batch through a radial bottom door. All the feeds and the discharge door were hand-operated by levers directly linked to the doors.

Long bogies, each of which carried six $\frac{1}{2}$-cubic-yard bottom-opening hoppers, were run beneath the batcher for loading and were drawn in pairs into the tunnel by battery locos. (A pair of such bogies is shown in *Fig. 4*.)

Near each main shutter five or six mixers were mounted; each mixer was served by a hoist for lifting the transport hoppers off the bogie and emptying the contents into the mixer drum. The outlet chute of each mixer discharged into a $\frac{1}{2}$-cubic-yard wet-mix hopper, which could be hoisted and tipped into a re-mixer mounted above the pump input. All this apparatus and the shutter carriages were mounted on the outer pair of four 2-foot-gauge tracks, leaving the inner pair free for traffic to the excavation face and for the shunting and unloading of the bogies. A pair of pumps and mixers, with their hoists, is shown in *Fig. 2*.

This arrangement had to be capable of producing an uninterrupted supply of concrete for periods of 30 hours and more in any weather conditions. A normal average output for one end of the tunnel was 30 cubic yards per hour, or about 1 batch per minute; the peak rate through one batcher was 90 batches per hour. About 39 semi-skilled operators were needed on each shift; a lapse on the part of any of them could cause

Fig. 1

View of Batching Plant at Eastern End of Tunnel

Fig. 2

Pair of Mixers, Hoists, and Concrete Pumps set up in Tunnel

Fig. 4

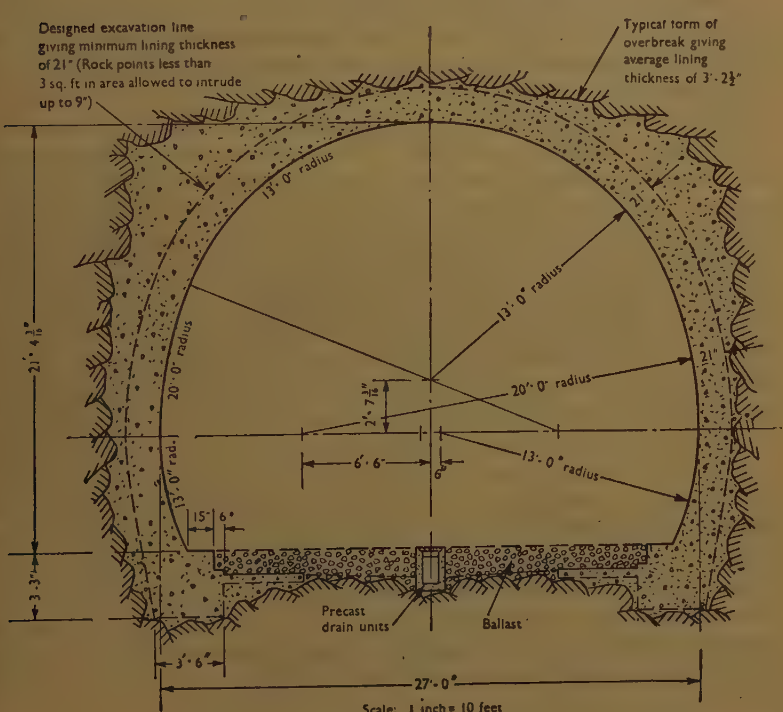
LENGTH OF COMPLETED LINING SHOWING HOPPERS TRANSPORTING DRY BATCHES ON LONG BOGIES

serious delay besides upsetting the concrete quality. Each "dry" batch had to pass through four constricted openings before mixing, at each of which sand and cement were liable to stick and build up. The weighing mechanisms were subject to rough treatment in the efforts to dislodge "stuck" parts of the batch quickly, and required frequent adjustment. This complexity of operation, combined with the need for sustained speed at every stage, set a practical limit to accuracy of control.

Pumping and Placing

As briefly described elsewhere,[1] the lining concrete had to be built up in one continuous operation between the skin of a steel shutter and the

Fig. 3



DIMENSIONS OF LINING STRUCTURE

irregular rock surface as excavated, in an arch-shaped section (*Fig. 3*) in lengths of 80 or 100 feet. It was vital that the whole space should be effectively filled, and as much as 900 cubic yards was sometimes required for the 100-foot shutter.

The pumps were of 4-inch nominal diameter (a size chosen for reasons of capital cost, economy, and availability), and each gave continous outputs of between 5 and 7 cubic yards per hour. By first building up concrete to crown level at the back end of the shutter (adjacent to the vertical stunt end of the last length filled), and then connecting the outlets of all pumps to valves in the crown as far back as possible, the concrete surface was kept to a minimum, and the risk of "cold joints" or of the disturbance of partially set concrete was minimized.

The success of the method depended upon the angle at which the concrete would stand in the 22-foot high annular space behind the shutter. If this angle became too flat, the area over which flow from the crown was taking place increased and delivery to the rearmost valves had to be continued so long that partial setting began near the valve itself. It was also possible, if the mix was not sufficiently stiff, for concrete which had been forced into the crown under pump pressure to drop away from the higher points when the nearby valves were closed and the pipes connected farther forward.

From another point of view, however, it appeared at first sight desirable to have as fluid a mix as possible. Three-quarters of the length of the tunnel was supported by steel ribs erected immediately after excavation, and positioned within the concrete lining with a minimum cover of 6 inches. Between the backs of these ribs and the rock surface steel packings had been thrust in, often very hurriedly, forming a random criss-cross pattern of old tram rails, scrap joists, and bank bars, and presenting a considerable obstacle to the complete filling of the overbreak voids. There was no possibility of removing this support system; everything had to be concreted in, and furthermore, in the crown and shoulders where the packing was greatest there was no chance of assisting flow by any form of vibration.

Tests were carried out; in these, crown packing conditions were simulated in enclosed boxes, and it was found that the pressure transmitted from a pump was sufficient to induce good penetration with any mix workable enough to pump, provided that the point of entry was not too far distant from the packing. With the workability chosen to give a good angle of flow, and which corresponded to a slump of 1½ inch, little trouble was experienced; both tell-tale devices worked from the crown of the shutter and the results of subsequent grouting indicated that the voids left were a very small percentage of the overbreak.

Materials

Aggregates could not be produced from local rock, and were brought about 70 miles from natural deposits of sand and gravel. The use of 4-inch diameter pumps limited the maximum size to ¾ inch. The quality was kept within the limits prescribed by B.S. 882, but it did not prove practicable to maintain a closer control of grading than is provided for in that specification. Experiments confirmed that the effect of grading

ations within the limits on workability at constant water/cement ratio, on strength at constant workability, was small compared with other rces of variation.

All cement was obtained from one modern works and gave consistent lts well above those specified in B.S. 12. The local water contained ing deleterious to concrete made with it.

kability

This was fixed within fairly narrow limits by the pumping and placing ria already described. The pumps could handle slumps down to it ½ inch, but, at this lower limit, blockages tended to result from any it variation. The 1½-inch slump chosen gave a reasonable working gin for pumping and, with this workability, the flow down the concrete ace behind the shutter took place without segregation. Below the lders, compaction was assisted by poker vibrators manipulated through ovable shutter panels; the crown and shoulders themselves compacted under the action of pump pressures. The finish achieved in this way own in *Fig. 4*.

ngth Margin

The minimum 28-day strength was fixed at 2,750 lb. per square inch; as necessary to estimate the margin by which average strength results d need to exceed this figure to prevent normal variations from giving ppreciable number of results below the limit. At the outset, there did seem to be any reason, with weigh-batching and the good materials able, why close control should not be achieved and a target average 400 lb. per square inch at 28 days was selected. This corresponded water/cement ratio of 0·66.

ing and Cement Content

Experiments in which ideal gradings were compared with the extremes e acceptable range confirmed that, so long as the ratio of coarse to aggregate was adjusted to keep the combined grading curve as near ossible to the centre of the permissible zone, the resulting mix was enough to the optimum for practical purposes. It then remained to the lowest cement content compatible with the required workability he water/cement ratio of 0·66. The cost of adding 1 lb. of cement to ½-yard batch worked out at about £600 for the whole job, a figure n was borne in mind when considering any addition to the strength in or average workability. The actual mix arrived at contained b. of cement per cubic yard of finished concrete; it was virtually 76 by volume, or 1 : 6·2 by weight. The coarse/fine ratio was ally 63 : 37 by weight, but this proportion was adjusted as sary to suit grading changes.

Operation

Quality was judged by the 28-day crushing strength of sample cubes, a minimum of three such cubes being made for each length of lining pumped. It was found that if three cubes were made from the same batch, the individual strengths differed from the mean of the group by only 1·4 per cent on the average and, accordingly, the best deployment of testing resources was obtained by sampling from a separate batch for each cube made. Although the average strength of these works cubes was, from the beginning, constant and close to the intended value, the variation of individual results was such that thirteen of the first seventy-five results fell below the minimum of 2,750 lb. per square inch.

Obviously it was not practicable to insist that the concrete represented by such cubes as fell below the minimum should be cut out and replaced; these results were simply reflecting a pattern of variation which would continue through all the batches not sampled. Normally, before pumping, a batch was thoroughly re-mixed with at least the last half of the preceding batch and the first half of the batch following it, and further mixing took place after delivery at the shutter. In order to arrive at the actual probability of a region of concrete in the lining being below standard, and to assess the causes of variation and the extent of the action needed to meet the designed quality limits, tests were run under working conditions; in these, samples of wet concrete at various stages were taken and analysed. In addition, extra cement tests were carried out and extra cubes were moulded from samples from re-mixers and from behind the shutter. Observations were made on the normal extent of errors affecting the batch weights and tests were run on individual pumps with special supervision at every stage.

Using the results of these tests and observations, it was estimated that if one batch in n were defective, the chance of a similarly defective region in the lining was not greater than 1 in n^2. Possible modes of failure of the thick continuous arch were considered, and it seemed clear that the structure would tend to develop its average strength before failure rather than to fail with the weakest patch, since transfer of stress from the weaker to the stronger portions of concrete appeared to be likely. The actual factor of safety of the lining represented by the first group of tests was considered to be within the required limits, but it was decided that the maximum allowable proportion of batches below 2,750 lb. per square inch should be limited to 8 per cent, and that special action should be called for if any result fell below 2,300 lb. per square inch.

The cause of the high variation was established as batching error; the make-up of the total standard deviation, S, of 793 lb. per square inch in the 28-day crushing strength of the first group was estimated as follows:

S_b (standard deviation due to batching errors): 730 lb. per square inch, approximately.

S_c (standard deviation due to cement quality variation): 240 lb. per square inch, approximately.

S_t (standard deviation due to testing and sampling variations): 200 lb. per square inch, approximately.

$$[S = \sqrt{(S_b^2 + S_c^2 + S_t^2)}]$$

It will be clear from the description of the batching and transporting system, and it was confirmed by the tests carried out, that random errors in weights of the solid constituents of each batch, when the batch reached the mixer, could be much higher than is usual with a weigh-batching system. There did not seem any prospect of getting aggregate and cement weights to within the 1 or 2 per cent which can be achieved in good conditions, but it was calculated that, assuming a "normal" distribution of results about the mean, the "8 per cent condition" could be satisfied without using a richer mix, provided that the standard deviation could be reduced from 804 lb. per square inch to 448 lb. per square inch, that is, from 23·8 per cent to 13·2 per cent of the average value of 3,378 lb. per square inch.

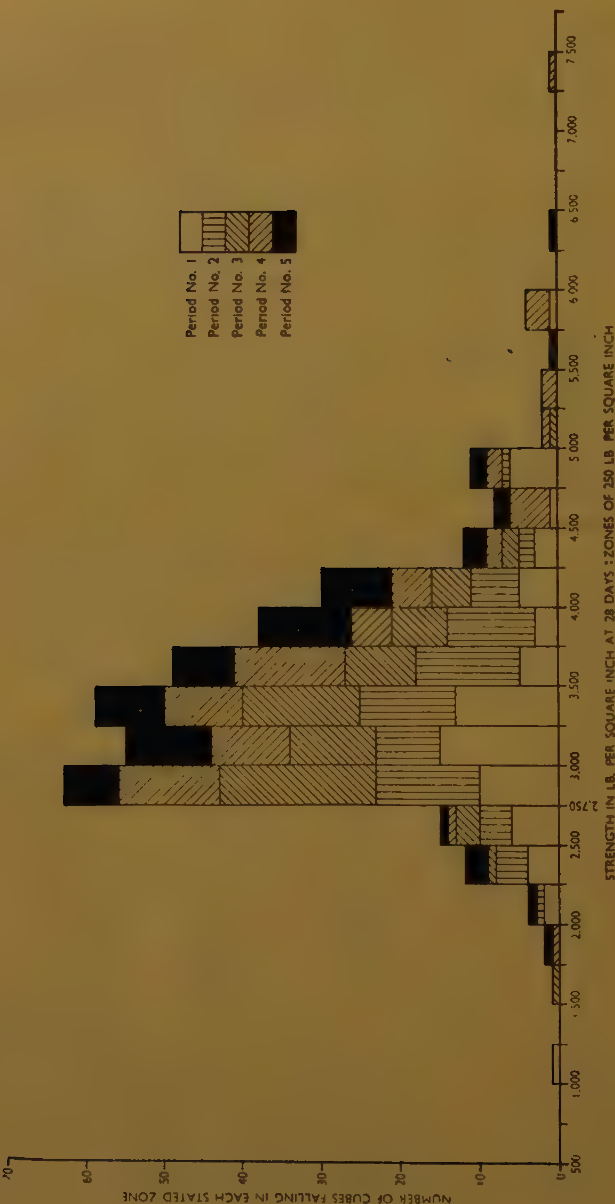
Opinions differed on whether this was a reasonable or indeed possible standard of control to demand, but it was resolved to try to achieve it. Considerable reduction in variation was achieved, and after the second period, the specified 8 per cent below 2,759 lb. per square inch was not appreciably exceeded. The results are summarized in Table 1.

TABLE 1

Period	Number of tests	Average strength: lb./sq. in. at 28 days	Standard deviation: lb./sq. in.	Results below 2,750 lb./sq. in.:	
				number	percentage
1	75	3,378	793	13	17·3
2	75	3,375	551	9	12·0
3	75	3,315	534	5	6·7
4	75	3,735	909	2	2·7
5	71	3,597	746	6	8·5

It will be noticed that the minimum strength criterion was achieved although the standard deviation was not reduced to the value calculated as being necessary for this object. In fact, with the average values and variations observed, the percentage below 2,750 lb. per square inch, for the five periods in order, should have been 21·7, 13·0, 14·5, 14·0, and 13·0 respectively. A glance at *Fig. 5*, on which the results for these periods are plotted on a histogram, will show that the assumption of "normal" distribution about the mean value is seriously misleading for this case.

Fig. 5



HISTOGRAM OF CUBE STRENGTHS

The physical reasons for this effect were quite simple. Cement was run out of the reservoir into the weigh-hopper after the coarse aggregate and before the sand, the flows being cut off by hand at marked points on a single dial, which showed the cumulative weight of material in the batch. The coarse aggregate was easy to stop cleanly at the right mark, but the cement often ran badly, so that extra cement in the batch was far more likely to result than the reverse. Cement consumption was checked by counting the bags emptied into the reservoir, and about 3 per cent more was consistently measured out in this way than was accounted for by the batch weights. Close observation showed that this extra was not spread evenly over a large number of batches, but affected isolated batches by as much as 15 per cent.

A second influence tending to give fewer results below rather than above the modal value was the action of inspectors at the mixers. Although attempts were made to keep the added water to the value calculated from the moisture-content determinations on the aggregates, the water gauges gave a lot of trouble, and the moisture contents were so variable in bad weather that it was impossible to keep up with the changes. The variation in solid content of the batches made judgement from workability difficult; a single batch could be seriously misleading. Nevertheless, positive instructions were given that any batch with a slump of more than 3 inches should be rejected, and the limit was later lowered to 2 inches. There is little doubt that this action resulted in the rejection of some sub-standard batches, without affecting the "balancing" above-average ones, and this increased the "skewness" of the histogram of cube strength results.

Results and Conclusions

The principal result to be recorded is that a large quantity of concrete was produced and placed rapidly under difficult conditions, with a smaller margin than is usual between the average strength and the effective minimum. Particular features of the operation are given below.

(1) *Control, Inspection, and Sampling*

These three functions were kept separate so far as possible. In so far as the contractor was held responsible for quality, no interference with his control of the processes affecting it could be allowed, although detailed instructions for every stage in concrete production were subject to discussion and agreement with the Resident Engineer before work began. After this, inspection was confined to observing that the agreed procedures were being carried out; if, after due warning, any observed irregularities were not corrected, the inspector's duty was to stop the job. The responsibility of measuring aggregate moisture-contents was not allowed to fall upon the inspectors, who merely checked readings taken by contractors'

engineers or foremen. However, inspectors did, perforce, have the power of veto against batches with excessive slump, and thus became to some extent the arbiters of *ad hoc* settings of the water gauges. Since there were sometimes thirteen mixers in action simultaneously and a shortage of skilled supervision, the help of inspectors at this point was an advantage, and led to no trouble.

Finally, it was recognized that the quality of the work depended on continuous supervision and inspection, of which sample tests provided, at best, an occasional accurate reflexion. Once the symptoms of variation in the batches produced—differing workabilities and pumpabilities, grading defects, etc.—had been correlated over the whole range with cube results, it was possible by visual inspection to gain a reliable impression of the pattern of variation in quality of the concrete going into the lining. The first function of the testing programme was therefore to give guidance for control and inspection. Thereafter it was sufficient if materials were checked regularly, and if sufficient concrete samples were tested at random to ensure that a statistically adequate group could be formed to correspond with any level of control observed. Control conditions remained tolerably constant for reasonable periods, and the practice of making three cubes per length (or roughly one in 400 batches) was found to give an adequate check on the variation in quality.

(2) *Recording, Plotting, and Analysis of Results*

The immediate plotting, in histogram form, of all results subject to variation proved to be a help in interpreting results and in establishing the pattern and limits of the varying quantities. A sufficiently accurate and very rapid method of computing standard deviation is indicated in Table 3 in the Appendix, in which the analysis of results from five representative periods is set out. Cube strength results were also plotted on time and location bases, to show any trend of results with these variables.

(3) *Plant*

The requirements at Woodhead, for the rapid production of small batches and the transport of dry materials after weighing, were no doubt unusual and exacting. It would probably have been justifiable from all points of view to have used more elaborate batching and transporting plant, with automatic operation wherever it could be introduced. The need for a simple, accurate, and foolproof water gauge was felt.

(4) *Cement Quality*

Indications of the effect of cement quality variation are shown in Table 2, part of the data for which was supplied by the manufacturers.

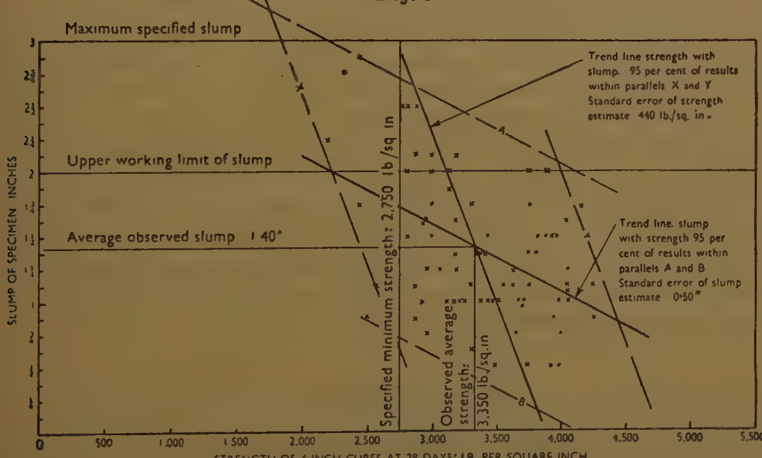
(5) *Slump as Control and Indication of Strength*

Despite its many disadvantages, slump was found to be the only

TABLE 2.—CEMENT TEST RESULTS

	Tensile tests (B.S. 12)				Compression tests on 4 : 2 : 1 : 0·6 vibrated concrete cubes
	Site		Works		
	3 days	7 days	3 days	7 days	28 days at works
Specified minimum : (lb. per sq. in.)	300	375	300	375	—
Observed average : (lb. per sq. in.)	476	526	613	662	3,973
Standard deviation : (lb. per sq. in.)	40·3	43·1	37·6	36·8	236
Coefficient of variation : (percentage)	8·5	8·2	6·1	5·6	6·0
Number of tests	97	90	209	208	39

Fig. 6



SLUMP–STRENGTH CORRELATION

property of wet concrete which could be measured frequently, simply, and by semi-skilled personnel. Slump-strength readings were plotted for various periods, and showed some correlation, as may be judged from *Fig. 6*, which is typical of all the sets of readings analysed. The limits to which control by slump (by prediction of strength from slump readings) could be taken, may be deduced.

(6) *General Conclusions*

The best combination of materials, placing organization, and quality control in concrete production may only be attained by co-operation between the designer, the site supervisor, the contractor, and the suppliers of plant and materials. A too rigid specification of mix at too early a stage may prejudice the striking of a balance between the different points of view. An increased consciousness of the saving in material possible when control methods are improved is likely to result in more attention being paid to control organization, and to the design of improved types of batching, handling, and mixing plant.

On the subject of the data available to assist in the estimation of variation in concrete quality, although information of a general application is badly needed, it is suggested that all general conclusions are, at present, premature. Far too little information has been produced, especially relating to average 28-day strengths of less than 3,500 lb. per square inch, for any degree of variation to be characterized as the result of good, bad, or indifferent quality control. In two respects the Woodhead results are out of line with recent generalizations.[2, 3] First, although standard deviation was reduced to about 500 lb. per square inch for a considerable time, which would be regarded as unusually good for an average of about 3,400 lb. per square inch, the control at these times was very far from ideal, and a different arrangement of plant could have reduced it very much further. Secondly, the unsymmetrical distribution of results upsets all theory based on the " normal " distribution curve. Until more information is available, it is suggested that every case must be referred to first principles, and the pattern of variations will need to be established by field trials.

If codes of quality control can be established, with a known relationship to production quality, and of application to standard types of plant, the designer will be better able to assess the true factor of safety, the contractor will be assisted in estimating costs, and the efficiency of use of concrete materials will be raised.

Acknowledgements

The Paper is presented by kind permission of the Civil Engineer, British Railways, Eastern Region, for whom the Woodhead new tunnel contract was carried out. Sir William Halcrow and Partners were consulting engineers for the works, and the Author is indebted to them for enabling him to write the Paper. Messrs Balfour, Beatty and Company were the contractors.

The Author wishes to thank Mr H. K. Arnold, Ph.D., B. Sc., A.M.I.C.E., for help in the preparation of the Paper.

The Paper is accompanied by three photographs and three sheets of

Zone centre	t	Period No. 1				Period No. 2				Period No. 3				Period No. 4				Period No. 5			
		f	ft	ft^2	$f(t+1)^2$	f	ft	ft^2	$f(t+1)^2$	f	ft	ft^2	$f(t+1)^2$	f	ft	ft^2	$f(t+1)^2$	f	ft	ft^2	$f(t+1)^2$
1,125	−9	1	−9	81	64	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1,375	−8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1,625	−7	0	0	0	0	0	0	0	0	1	−7	49	36	0	0	0	0	0	0	0	0
1,875	−6	0	0	0	0	0	0	0	0	0	0	0	0	1	−6	36	25	1	−6	36	25
2,125	−5	2	−10	50	32	1	−5	25	16	0	0	0	0	0	0	0	0	1	−5	25	16
2,375	−4	4	−16	64	36	4	−16	64	36	1	−4	16	9	0	0	0	0	3	−12	48	27
2,625	−3	6	−18	54	24	4	−12	36	16	3	−9	27	12	1	−3	9	4	1	−3	9	4
2,875	−2	10	−20	40	10	13	−26	52	13	20	−40	80	20	13	−26	52	13	7	−14	28	7
3,125	−1	15	−15	15	0	8	−8	8	0	11	−11	11	0	10	−10	10	0	11	−11	11	0
3,375	0	13	0	0	13	12	0	0	12	15	0	0	15	10	0	0	10	9	0	0	9
3,625	1	5	5	5	20	13	13	13	52	9	9	9	36	14	14	14	56	8	8	8	32
3,875	2	3	6	12	27	11	22	44	99	7	14	28	63	5	10	20	45	12	24	48	108
4,125	3	5	15	45	80	6	18	54	96	5	15	45	80	5	15	45	80	9	27	81	144
4,375	4	3	12	48	75	2	8	32	50	2	8	32	50	2	8	32	50	3	12	48	75
4,625	5	1	5	25	36	0	0	0	0	0	0	0	0	5	25	125	180	2	10	50	72
4,875	6	6	36	216	294	1	6	36	49	0	0	0	0	2	12	72	98	2	12	72	98
5,125	7	0	0	0	0	0	0	0	0	1	7	49	64	1	7	49	64	0	0	0	0
5,375	8	0	0	0	0	0	0	0	0	0	0	0	0	2	16	128	162	0	0	0	0
5,625	9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	81	100
5,875	10	1	10	100	121	0	0	0	0	0	0	0	0	3	30	300	363	0	0	0	0
6,125	11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6,375	12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	12	144	169
6,625	13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6,875	14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7,125	15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7,375	16	0	0	0	0	0	0	0	0	0	0	0	0	1	16	256	289	0	0	0	0
Sums	Σ	75	+1	755	832	75	0	364	439	75	−18	346	385	75	108	1,148	1,439	71	63	689	886
Period results		$\bar{x} = 3{,}378$ $s = 793$				$\bar{x} = 3{,}375$ $s = 551$				$\bar{x} = 3{,}315$ $s = 534$				$\bar{x} = 3{,}735$ $s = 909$				$\bar{x} = 3{,}597$ $s = 746$			
Cumulative results		$\bar{x} = 3{,}378$ $s = 793$				$\bar{x} = 3{,}377$ $s = 683$				$\bar{x} = 3{,}356$ $s = 638$				$\bar{x} = 3{,}451$ $s = 734$				$\bar{x} = 3{,}479$ $s = 738$			

drawings, from which the half-tone page plates and the Figures in the text have been prepared, and by the following Appendix.

Appendix

Approximate values of standard deviation for selected periods were obtained rapidly by the following method.

Results for the period were first plotted in the histogram form (*Fig. 5*). The strength zones of this histogram were then numbered outwards from a zero containing the estimated mean strength. Table 3 was then drawn up, showing the number, f, of results against the zone number, t, and the strength representing the centre of the zone. The columns ft^1, ft^2, and $f(t+1)^2$ were then completed, and each column was totalled to give Σf, Σft, Σft^2, and $\Sigma f(t+1)^2$. Totals were checked from the equation:

$$\Sigma f(t+1)^2 = \Sigma ft^2 + 2\Sigma ft + \Sigma f$$

Mean strength $\bar{x}$ is derived from the formula:

$$\bar{x} = x_0 + c\,\frac{\Sigma ft}{\Sigma f}$$

where x_0 denotes zone centre of zero zone and c denotes the width of each zone.

Standard deviation, S, is given approximately by:

$$S = c\sqrt{\frac{\Sigma ft^2}{\Sigma f} - \left[\frac{\Sigma ft}{\Sigma f}\right]^2}$$

Reeerences

1. P. A. Scott and J. I. Campbell, "Woodhead New Tunnel: Construction of a Three-Mile Main Double-Line Railway Tunnel." See p. 506.
2. L. J. Murdock, "The Control of Concrete Quality." Proc. Instn Civ. Engrs, Part I, vol. 2, p. 426 (July 1953).
3. F. R. Himsworth, "The Variability of Concrete and its Effect on Mix Design." Proc. Instn Civ. Engrs, Part I, vol. 3, p. 163 (March 1954).

ORDINARY MEETING

27 April, 1954

DAVID MOWAT WATSON, B.Sc.,
Vice-President, in the Chair

The Chairman, after asking the members present to stand, announced with deep regret the sudden death, on Friday, 23 April—on the eve of his retirement from the secretaryship—of the secretary, Mr E. Graham Clark. He had died in a nursing home at Eastbourne after a short illness. The Chairman stated that at its meeting during the afternoon the Council had passed a resolution of condolence.

It was resolved :—

"That Messrs H. M. Bostandji, H. R. Boyce, D. A. Brown, Robert Carey, E. W. Cuthbert, R. W. A. Fane, Andrew Henderson, and E. C. Lightbody be appointed to act as scrutineers, in accordance with the By-Laws, of the Ballot for the election of the Council for the year 1954–1955."

The Council reported that they had recently transferred to the class of

Members

Atkinson, William Harold, B.Sc.(Eng.) (*London*).
Brass, *Lt-Col* John Henry, R.E.
Clerke, Robert William Goodwin.
Davies, Hugh Vaughan.
Dunn, Dermot John, M.A., B.A.I. (*Dublin*).
Osborn, Dryden Frank, B.Sc. (*Witwatersrand*).
Parker, John, B.Sc. (*Durham*).

and had admitted as

Graduates

Attygalle, Senagi, B.Sc.(Eng.) (*London*), Stud. I.C.E.
Barrey, Alan Erskine, B.Sc. (*Durham*), Stud.I.C.E.
Beynon, Glyndwr Ian Malcolm Treharne, B.Sc.(Eng.) (*London*).
Bickley, John Aubrey, Stud.I.C.E.
Blower, Robert Wingfield, B.Sc. (Eng.) (*London*).
Boston, Ronald Merrick.
Bowman, Robert Cairns, B.Sc. (*Glasgow*), Stud.I.C.E.
Chatterjee, Binoy Kumar, M.Sc.(Eng.) (*London*).
Chilver, Amos Henry, B.Sc. (*Bristol*).
Coldwells, Clement Arthur.
Collins, Philip John, B.Sc.(Eng.) (*London*).
Cottell, Michael Norman Tizard, Stud.I.C.E.
Denman, Roger Darrel, B.Sc. (*Leeds*), Stud.I.C.E.
Donald, Laurence, B.Sc. (*Aberdeen*).
Doxey, Francis William, B.Sc.(Eng.) (*London*).
Du Heaume, Peter Learmonth, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Eddershaw, Basil Wainwright, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Eddie, Alastair George Ferguson, B.A. (*Cantab*.).
Emmett, Robert, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Evans, Trefor Jenkins, B.Sc. (*Wales*).
Evans, Vivian Harvard, B.Sc. (*Wales*), Stud.I.C.E.

Exelby, Richard Edmund, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Fenney, Eric, B.Eng. (*Sheffield*), Stud. I.C.E.
Ferguson, Edward Reginald William, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Gow, Alistair Ian Stuart, B.Sc. (*Aberdeen*).
Graham, Derrick Austin, B.Sc. (*Durham*), Stud.I.C.E.
Gray, Robert Stevenson, Stud.I.C.E.
Green, Denis Hall, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Greenwood, David Anthony, B.Sc. (*Leeds*), Stud.I.C.E.
Hanrahan, Michael Francis, B.E. (*National*), Stud.I.C.E.
Heaver, James Robert, Stud.I.C.E.
Hemingway, Geoffrey Truscott, B.Sc. (*Cape Town*).
Hornby, Ivor Wilfred, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hosking, Claude Henry, Stud.I.C.E.
Jude, Dennis Victor, B.Sc.(Eng.) (*London*).
Kelly, Kevin Denis, B.E. (*New Zealand*).
Klein, Richard Lawrence, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Leonard, John Michael, B.A., B.A.I. (*Dublin*).
Littlewood, Alan, B.Sc. (*Manchester*), Stud.I.C.E.
Locking, Edgar Reed, B.Sc.(Eng.) (*London*).
Luetchford, Michael Alfred Cecil, B.Sc.(Eng.) (*London*), Stud.I.C.E.
McCallum, George Hugh.
McDowell, Charles William Michael, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Mackay, Coinneach Ian, B.Sc. (*Aberdeen*).
Meyer, Benjamin, B.Sc.(Eng.) (*London*).
Moore, John, Stud.I.C.E.
Morgan, Peter Lewys Thomas, Stud. I.C.E.
Murray, Gordon Stewart, Stud.I.C.E.
Oates, Raymond Kenneth, Stud.I.C.E.
Phelps, Harry Orville, B.Sc. (*Wales*), Stud.I.C.E.
Philcox, Keith Thomas, Stud.I.C.E.
Prangnell, Kenneth John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Pullen, Robert Hulme, B.Sc.(Eng.) (*London*).
Rees, Geraint, B.Sc. (*Wales*).
Scott, Ian Leslie, B.Sc. (*St. Andrews*).
Sharp, Robert Frank.
Shreeve, Dennis, B.Sc.(Eng.) (*London*).
Skinner, Richard Newlyn, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Smith, Roger Arnold, B.E. (*Sydney*).
Smyth, William Norman, B.Sc. (*Belfast*).
Speed, John Jeffery, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Spratt, Barrie Hugh, Stud.I.C.E.
Thompson, Douglas Browne, Stud. I.C.E.
Tregear, Keith, Stud.I.C.E.
Varley, William Raymond, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Walker, James Maurice, B.A., B.A.I. (*Dublin*).
Walker, John Grenville, *B.E.M.*, B.Eng. (*Liverpool*), Stud.I.C.E.
Wanigasekara-Mohotti, Don Padmatilleke, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Watson, Philip James, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Watt, Colin John George, B.Sc. (*Aberdeen*).
Wilson, Keith Francis Denis, B.Sc. (Eng.) (*London*).
Woolley, Malcolm Victor, B.Sc. (*Birmingham*), Stud.I.C.E.

and had admitted as

Students

Al-Kamil, Abdul Karim Maki.
Bakker, Jaap Jelle.
Bell, John Arthur.
Briggs, John Reginald.
Cottee, John Geoffrey.
Criticos, Demetre Mazarakis.
Dick, John Jack.
Douglas, Daniel McDonald.
Edwards, John Anthony Cromar.
Elliott, Christopher Dennis.
Evans, Maldwyn.
Everett, Cyril Frank.
Findlay, Peter James.
Ford, Charles Rostron.
Garner, Peter Ronald.
Gird, Desmond Gustav.
Green, Roger.
Green-Armytage, Keith.
Hagenbach, Paul.
Hamilton, John McCulloch.
Handley, Jack.
Hayes, Harold Ian.
Hobbs, John Michael.
Huang, John.
Hunter, Hewison.
Hutchinson, Bruce Brooks.

Knapman, Roger John.
Langford, James Edward.
Law, James Richard.
Lee Pak Fong.
Lewis, John Reginald Jones.
Lidiker, Stanley.
Livingston, Donald.
McLullich, Malcolm.
McPhee, James Gordon.
Malik, Mohammad Hayat.
Marr, Ben Emslie.
Marshall, Alexander Ludovic.
May, Ronald.
Miano, Eliphelet.
More, George Eric.
Morrison, Robert William Mackenzie.
Nancarrow, David Rodney.
Niven, Robert Kemp.
O'Flaherty, Coleman Anthony.
Okoye, Peter Ikemefunah.
Panas, Gorassimos Evangelos Vassilios.
Pargeter, Brian.
Parkinson, Joseph Maurice.
Parry, Malcolm Frederick.
Potter, Uter Alan.
Powell, Joseph Brian.
Price, Nigel John.
Radcliffe, Dennis Richard.
Robson, John Paul.
Routledge, Robert.
Russell, Robert Lawson.
Shaw, Douglas Perston.
Shearer, James Barrie.
Sides, Geoffrey Raymond.
Stanyon, Kenneth Albert.
Steuart, Keith Alexander.
Storch, Barry.
Stutchbury, Richard Charles.
Taylor, Harry.
Taylor, James Peter.
Thomas, Bryan.
Timmins, John.
Vallance, Joseph James Frederick.
Van Schalkwyk, Andries Johannes.
Vickers, Hugh William Hayne.
Ward, Walter George.
Wheatley, James Harold.
Wright, Tony Albert.

JAMES FORREST LECTURE, 1954

The Chairman said that the James Forrest Lectures had been established in 1891 at the wish of James Forrest, who was Secretary of the Institution from 1859 to 1896, and Honorary Secretary until his death in 1917.

The original endowment had been the balance of a sum of money subscribed by members of the Institution for engraving the portrait of Mr Forrest; to that endowment Mr Forrest had added a similar sum by bequest to the Institution, in order to establish a series of Lectures. He had also bequeathed to the Institution a number of pieces of presentation silver, of which he had been the recipient during his Secretaryship.

The evening marked the sixtieth Lecture of the series. Dr W. J. Pugh had had a long and distinguished career as a geologist, having been Professor of Geology at the University of Wales and, later, at the University of Manchester. He had been the Deputy-Vice-Chancellor at Manchester University from 1943 to 1950. Since 1950 he had been the Director of the Geological Survey of Great Britain and Museum of Practical Geology and, since 1951, he had been Emeritus Professor of Geology of the University of Manchester. He had been a Member of the Council of the Royal Society in 1952.

Dr Pugh had taken as the subject for his Lecture, "The Geological Survey of Great Britain."

Dr Pugh then delivered his Lecture.

"The Geological Survey of Great Britain"

by

William John Pugh, O.B.E., D.Sc., B.A., M.I.M.E., F.G.S., F.R.S.

I MUST first thank you for the honour which you have conferred upon me by inviting me to give the James Forrest Lecture, and particularly in asking me to speak about the history and work of the Geological Survey of Great Britain. It is appropriate that I should commence with a few words about the setting in which the Survey began its work more than 100 years ago.

The close of the eighteenth and the beginning of the nineteenth centuries was a historic period in the development of geological science. James Hutton (1726–1797) discussed the formation of rocks and showed that the phenomena exhibited by them might be explained by causes operating at the present time. His " Theory of the Earth " was published in 1795 and he may be regarded as the founder of modern geology. William Smith (1769–1839) discovered that different fauna fossils distinguish strata of different ages. He was responsible for the study of the order of age and

arrangement of the sedimentary rocks, thereby laying the foundations of stratigraphical geology. His geological map of southern Britain, an immense step forward, was published in 1815. Charles Lyell (1797–1875) published the first volume of his great book, "The Principles of Geology," in 1830 and set geology upon its present course as a systematic science. The principles of geology had been established and expounded; there was a new approach to the study of geological structure and history.

The most important method of recording geological information and of making known geological discoveries is the geological map. It was not possible to prepare geological maps as we now understand them until William Smith had established the principles of the stratigraphical classification of rocks; he was the first to make a geological map on truly scientific lines. He had made a map of the Bath country as early as 1799, but his masterpiece was the coloured map on the scale of 5 miles to the inch of England, Wales, and part of Scotland published in 1815. G. B. Greenough, the first President of the Geological Society of London, began the preparation of a map of England and Wales in 1808 which was published in 1820 on the scale of 6 miles to the inch. Progress was also being made in Scotland and in Ireland; John Macculloch's map of Scotland on the scale of 4 miles to the inch was published in 1836 and Richard Griffith's map of Ireland on the same scale in 1839. In addition to these general maps, others covering local areas appeared in books and in the journals of learned societies. The preparation of geological maps was evidently making considerable progress, but one of the greatest difficulties at this time was to obtain suitable topographical maps upon which to record geological observations. William Smith, for example, had to record his observations directly upon a map on the scale of 5 miles to the inch.

The Ordnance Survey (at first known as the Trigonometrical Survey) was officially established in 1791. Topographical surveying was almost entirely confined to the scale of one inch to the mile until 1824 by which time most of southern England and part of Wales had been covered. Then the 6-inch survey of Ireland was commenced, but similar work in Great Britain did not begin until 1840 in the north of England and the south of Scotland; it was not until 1870 that one-inch sheets were available for the whole of Great Britain. Surveying on the scale of 25 inches to the mile began in 1854.

In 1826 the Ordnance Survey came under the control of Captain Colby who was anxious that his topographical maps should be used for geological surveys, and in 1832 Henry de la Beche was authorized to place colours illustrative of the geology upon the one-inch sheets of south-west England. Three years later, the Ordnance Survey sought the advice of the President of the Geological Society of London and the Professors of Geology at Oxford and Cambridge, who were also prominent members of that society, upon the best way of combining the topographical and geological mapping. De la Beche was chosen to direct and organize the geological work and thus

was founded the Geological Survey of Great Britain. The Geological Survey may be regarded as the child of the Ordnance Survey and the Geological Society of London. In a wider setting it grew out of the quickening of geological thought and the development of topographical mapping at the beginning of the nineteenth century.

For 4 years, De la Beche was the only officer of the Survey but he completed his mapping of south-west England, and in 1839 published a comprehensive report on this part of the country, the first memoir of the Geological Survey. He founded the Museum of Practical Geology (at first known as the Museum of Economic Geology) which is still an integral part of the Survey, the first School of Mines in this country which is now the Royal School of Mines and part of the Imperial College of Science and Technology, and a Mining Records Office which was later transferred to the Home Office. An epoch in geological mapping began under his direction; in addition, members of his staff carried the gospel to other parts of the British Commonwealth where geological surveys were established on the pattern of that in Great Britain.

The Organization and Functions of the Geological Survey and Museum

The Geological Survey and Museum are under the control of the Department of Scientific and Industrial Research. The headquarters office and the museum are in South Kensington and there are branch offices in Manchester, Newcastle, Edinburgh, and Belfast. The scientific staff is divided into two groups—field and special. The field staff is concerned primarily with geological mapping and is allocated to the eight regions into which at present Great Britain is divided for that purpose; to those must be added Northern Ireland. The special staff is concerned essentially with special branches of geology, namely, petrology, palaeontology, geophysics, water, atomic energy, and the Museum.

The functions of the Survey may be summarized as follows:—

(1) The preparation and publication of geological maps and memoirs of Great Britain and Northern Ireland.

(2) The investigation of the geology of coal, iron-ore, non-ferrous ores, minerals, and rocks used for industrial purposes.

(3) The investigation of problems of water supply, particularly of underground water.

(4) The discovery and evaluation of raw materials, at home and overseas for atomic energy.

(5) The maintenance and development of the Museum of Practical Geology as a centre for instruction and research.

(6) The provision of geological information and advice to Government departments, national boards, industrial concerns, and the general public.

The Preparation and Publication of Geological Maps and Memoirs of Great Britain and Northern Ireland

The Geological Survey was founded to prepare copies of the Ordnance Survey maps geologically coloured in order to provide an accurate representation of the geology of this country which would be of service in all branches of industry. Its primary function is still the preparation and publication of geological maps and memoirs. The purpose of these maps and memoirs is to provide information upon all aspects of British geology. They are the basis of all its activities and the most important service the Survey can give to science and to industry.

When the Geological Survey was founded, the only Ordnance Survey maps which were available were on the scale of one mile to the inch and geological mapping, therefore, began upon them ; the whole of Wales and practically the whole of southern and central England was so mapped. Mapping on the scale of 6 inches to the mile began in England in 1860 in the Lancashire coalfield and was gradually extended over most of northern England. The one-inch geological maps which were produced are now called the Old Series and they were available for practically the whole of England and Wales by about 1884.

When the New Series Ordnance Survey maps appeared for England and Wales, they were adopted for the one-inch geological maps and these New Series geological sheets, in all cases constructed from 6-inch mapping, began to be issued in 1893. They are usually published in two editions, Drift and Solid ; the former shows the superficial deposits, for example, of glacial origin and the solid rocks where they are exposed at the surface ; the latter the solid rocks as they would appear if the cover of superficial deposits were removed. There are still, however, large areas which are not covered by the New Series maps.

The mapping of Scotland, which commenced in the Lothians in 1854, was carried out partly on the one-inch but mainly on the 6-inch scale. Work north of the Highland border did not begin until 1877 and, in the north-west Highlands, until 1882 ; a considerable area in Inverness-shire and the whole of the Outer Hebrides have yet to be systematically mapped by the Geological Survey.

The Survey commenced mapping in Ireland as early as 1845 and, since 6-inch topographical maps were available, the work was undertaken from the outset on that scale. The one-inch geological maps of Ireland began to appear in 1855 and the survey was completed in 1887 although many of the sheets were not published until after that date. Responsibility for official geological work in Ireland was taken out of the hands of the Survey in 1905. In 1947 at the request of the Government of Northern Ireland, the Survey established an office in Belfast to carry out geological work in that country.

The Geological Survey today carries out its mapping in all areas on the

6-inch scale unless a still larger one is required for some particular purpose. The programme falls into two categories; primary mapping where ground is being covered for the first time on the 6-inch scale, and revision mapping where existing 6-inch maps are being brought up to date in the light of new information and new knowledge.

The area still to be surveyed on the 6-inch scale is approximately 22,000 square miles or about 40 per cent of England and Wales and about 3,000 square miles or nearly 10 per cent of Scotland; however, a great deal of work has been done in these areas by geologists outside the Survey. Nevertheless, the completion of the 6-inch survey of Great Britain is a matter of real importance, providing as it will fundamental information upon the geology of this country.

Revision mapping is necessary in all areas, especially in those of economic importance, so that new information derived from quarrying, mining, and boring may be collected leading to a fuller understanding of the geological conditions. Geological mapping is not static; new techniques and new knowledge demand constant reconsideration of areas upon whatever scale they may have been mapped. There is no finality in the collecting of geological information. More and better geological maps are needed both for scientific and industrial purposes. Mapping must always remain the primary function of the Geological Survey.

The one-inch Ordnance Survey sheets are the units for the programme of primary and revision mapping. Primary mapping is at present in progress on the igneous and metamorphic rocks of the Scottish Highlands north of Fort William, on the Carboniferous rocks of the Southern Pennines, in the west Midlands in the classic country of Pre-Cambrian and Lower Palaeozoic rocks around Church Stretton and farther east in the Jurassic ironstone country in Lincolnshire, in the west country on the Carboniferous, Triassic, and younger rocks around Bristol and Wells, on the Jurassic and Cretaceous rocks north of Cambridge and the Cretaceous and Tertiary sediments of the Canterbury district, and in all these regions upon the superficial deposits.

The primary survey of the Bristol–Somerset coalfield has recently been completed and a special map is in preparation covering more than 600 square miles. Revision mapping is in progress in many other coalfields, in the South Wales, Lancashire, Yorkshire, and Durham coalfields, across the Border in the Canonbie, Midlothian, and Central coalfields of Scotland, and in Northern Ireland in the Dungannon and Ballycastle districts; in all the coalfields a careful watch is maintained for new information made available by mining and boring.

A geological map shows the distribution and arrangement of the rocks at the surface of the ground. It is a two-dimensional picture of phenomena which possess three dimensions. The third dimension of underground distribution and arrangement can, in general, be inferred from the map and illustrated by horizontal and vertical sections. This third dimension is of

great scientific and economic importance and the inferences concerning it are supplemented wherever possible by direct information from mines and boreholes. It is, therefore, appropriate to mention that, by the Mining Industry Act of 1926, it is a statutory obligation to notify the Geological Survey of all sinkings and borings to a depth of more than 100 feet for minerals and by the Water Acts of 1945 and 1946 to more than 50 feet for water. The notifications under the Mining Industry Act in Great Britain in 1953 numbered 740 and under the Water Acts 1,047. A very large amount of valuable underground information is thus collected; for example, under the Mining Industry Act in 1953 the Survey examined a total of more than 270,000 footage or about 50 miles. The underground examination of rocks exposed in tunnels, cross-measure drifts, and so forth, dealt with more than 61,000 feet in England and Wales (of which more than 46,000 feet were in colliery workings) and with more than 67,000 feet in Scotland (mainly in tunnels driven by the North of Scotland Hydro-Electric Board).

There are, however, many variables of succession, structure, and history which set severe limitations upon inferences concerning the underground arrangement of the rocks in the absence of direct information from mines and boreholes; this is particularly true with increasing depth below the surface. This is the main reason for the use of geophysical surveys in association with geological surveys and also for undertaking a special programme of boring.

Geophysical surveys are an important way of investigating underground structure, the main methods being gravitational, magnetic, electrical, and seismic. The Geological Survey has been interested in geophysics for many years but has undertaken systematic regional investigations only since the war. It is at present mainly engaged upon work by gravimeter and magnetometer in the English Midlands and in Northern Ireland; in the former, as part of a systematic survey commenced in 1950 which is yielding interesting and suggestive results, and in the latter to assist in the elucidation of an area which is largely concealed by superficial deposits. Gravitational and other work is also being carried out by many other investigators and the Survey is collecting and collating the gravity data for the whole of Great Britain in order to publish gravity maps as transparent overlays to the geological maps with a view to gaining a fuller understanding of underground structure.

The results of geophysical surveys are often difficult to interpret and usually need to be tested by drilling boreholes, but on the other hand such surveys are valuable in suggesting sites for exploratory boreholes. A gravity survey carried out by the D'Arcy Exploration Co. Ltd across south-central England was published in 1951. This showed a pronounced gravity "low" in several places; that over the estuary of the Thames seemed to warrant investigation by a borehole since it might possibly represent a basin of coal-bearing rocks although such evidence as was

available from boreholes in neighbouring areas was not wholly favourable. The Geological Survey undertook in 1953 a deep boring in Canvey Island. This passed through the cover of younger Mesozoic rocks and, as anticipated, entered the older Palaeozoic rocks at 1,317 feet below surface, but those older rocks were of pre-Carboniferous age.

Seismic reflexion work carried out a few years ago by Sir Edward Bullard and others indicated that the surface of the Palaeozoic floor beneath the Mesozoic was probably about 438 feet below surface at Cambridge. Professor W. B. R. King initiated a scheme to investigate the depth to, and the nature of, the basement by boring. The Royal Society and certain industrial firms made grants towards the cost of the borehole and ultimately the project was undertaken by the Survey. The borehole, which will be kept permanently open to provide opportunities for instruction and research in the University, entered the Palaeozoic at almost exactly the same depth inferred from the seismic survey but the floor was quite unexpectedly found to be Carboniferous limestone.

It may be mentioned that valuable geophysical work can be carried out in boreholes by lowering equipment into them and recording on meters various kinds of physical information, for example, gamma radiation and electrical variations. Gamma radiation and electrical logs are related to the nature of the rocks at different levels; they are, therefore, useful where solid cores are not taken and as a means of correlating the rocks in different boreholes.

Since 1945 the Geological Survey has undertaken a programme of boring in order to investigate the underground structure of areas where there is a lack of direct information; two examples in association with geophysical surveys have been given but there are others. We have recently completed the 6-inch mapping of the Bristol–Somerset coalfield. It so happens that the strata between the Carboniferous limestone and the Coal Measures are very poorly exposed, being largely concealed beneath Triassic rocks. A borehole was drilled in 1953 in the southern outskirts of Bristol to a depth of 2,195 feet. It began in Trias and passed, as expected, through the lower part of the Coal Measures, then through the previously unknown rocks, several hundred feet thick and roughly equivalent to the Millstone Grit, eventually reaching well-known levels in the Carboniferous Limestone Series. The information so obtained is of great value in completing our knowledge of the Carboniferous rocks and also, if these previously unknown rocks are encountered, it should now be possible to forecast the position of the coal-bearing strata overlying them.

In 1875 a borehole near Burford in Oxfordshire was said to have entered Coal Measures at depth but the records were inconclusive. A later borehole at Batsford, about 16 miles north of Burford, proved 524 feet of barren Upper Coal Measures beneath the Mesozoic and resting on Silurian. It was decided in 1953 to drill a deep hole near Burford. This passed through the Mesozoic cover and, as expected, entered Coal Measured at a depth of

1,107 feet. It proved a thickness of 2,660 feet of these non-productive measures with only occasional thin coal seams. Unfortunately, at a depth of 3,767 feet drilling had to be stopped because of technical difficulties and the total thickness of the Upper Coal Measures is not known, nor whether they are underlain by productive Coal Measures.

The Survey is at present carrying out several shallow borings to a depth of 500 feet or less to assist in the construction of the geological map of the Stockport area. The primary 6-inch mapping has been recently completed but over extensive areas the solid rocks are entirely concealed beneath a thick mantle of glacial deposits. A series of boreholes will pierce these superficial deposits and for some distance descend into the underlying rocks. It is hoped that the information so obtained will enable the position and nature of certain faults to be fixed more precisely and the solid geology to be more accurately shown on the new one-inch map.

The work of the field geologists is supplemented in a vital way by the geologists working in the special departments, particularly, for example, in petrology and palaeontology. The petrographical department applies the appropriate laboratory techniques in the examination and identification of rocks and minerals which have been collected, especially in thin section under the microscope. The palaeontological department examines and identifies the fossils so essential as markers determining different levels in the succession of stratified rocks. These departments thus play their parts in the accurate depiction of the geology upon the maps and of its description in the memoirs. The completion of the maps and memoirs is a combined operation of the field and specialist geologists.

It is convenient to mention here the maps which are published by the Survey. They are: the 6-inch maps of the coal-fields and of the London district (but any completed 6-inch map is available for consultation), and the one-inch maps, which in turn are reduced to those on the scales of 4, 10, and 25 miles to the inch, all colour printed, and giving a progressively generalized picture of the geology of Great Britain. The memoirs are usually explanatory of the one-inch sheets but there are special memoirs and monographs in varied fields of geological science. These maps and memoirs which have been published for more than 100 years provide fundamental information upon most aspects of British geology.

The Investigation of the Geology of Coal, Iron-ore, Non-ferrous Ores, Minerals, and Rocks used for Industrial Purposes

The discovery, development, and exploitation of the mineral resources of a country depend in the first instance upon a knowledge of its geology; it is a function of the Survey to provide that knowledge. There is a great variety of naturally occurring materials of economic importance in Great Britain but coal and iron-ore are our staple mineral resources.

Coal.—Coal is the most important mineral in Britain and our industrial

development is based upon it. The output, the third largest of any country in the world, is more than 200 million tons. The Geological Survey is greatly concerned with the coalfields and its responsibilities in respect of them have increased since the unification in 1947 of the coalmining industry under the National Coal Board. The Survey has an important part to play in providing geological information and advice both in respect of current production and in planning for the future.

A year or two ago, the Survey published a new edition of its memoir on the concealed coalfield of Yorkshire and Nottinghamshire and this East Pennine field may be taken as illustrative of its work. It is the largest and most productive in Britain; the proved area covers about 3,000 square miles and the production is more than a third of our total output. It is a partly exposed and partly concealed coalfield; the exposed portion is in full production, the concealed portion is partly developed, but has become one of the most important coal-mining areas in Britain.

The exposed portion has been closely investigated by mapping the rocks at the surface, supplemented wherever possible by information derived from mining. The detailed succession, including the position and thickness of the coal seams, and their structural arrangement has been determined as precisely as possible and all this has been recorded on maps illustrated by vertical and horizontal sections.

The investigation of the concealed portion presents new problems since there the older coal-bearing rocks are beneath and wholly concealed by an unconformable cover of younger rocks. Moreover, the structure of the older Carboniferous rocks, including the Coal Measures, is quite different from that of the younger Permo-Triassic rocks; the former are folded and faulted, the latter rest discordantly upon them and are inclined eastwards at comparatively low angles. This general relationship might be forecast from the study of the exposed coalfield and its margins, but detailed knowledge now depends upon information from mines and boreholes.

Exploration and exploitation have been active in many collieries near the exposed Coal Measures and extensive boring has been undertaken in more easterly areas. In addition, in Nottinghamshire three small oilfields have been in production since their discovery between 1939 and 1943 and a very large number of holes have been drilled in connexion with them. All this underground investigation has greatly increased our knowledge of the concealed coalfield which is now proved to extend eastwards and to cover at least 2,000 square miles.

The Coal Measures in the concealed field consist of the productive Grey Measure, up to 5,000 feet of grey mudstones and sandstones with coal seams which are overlain in Nottinghamshire and Lincolnshire by the barren Red Measures, and up to 600 feet of red or vari-coloured mudstones and sandstones without coals. The productive measures consist of a succession of minor subdivisions, each one comprising a group of strata deposited in one sedimentary cycle, the normal upward succession being coal, mudstone,

sandstone, and seat earth with, say, an average thickness of 30 to 40 feet. The lower part of some of these cycles, which are not always complete, may contain marine fossils and these marine bands are of great value in correlating the coal seams of one area with those of another and in determining stratigraphical levels in boreholes.

The succession in the Coal Measures has been investigated in great detail and a zonal sequence, based on the occurrence of non-marine lamellibranchs, has been established as in other British coalfields. The variations in character and thickness of the subdivisions have been portrayed in vertical sections and isopachyte maps ; similar sections and maps have been prepared for individual coal seams.

The Carboniferous rocks were not only folded and faulted, but also uplifted and denuded before the Permo-Triassic rocks were deposited unconformably upon them ; consequently, different levels in the Coal Measures "crop out" against the base of the unconformable cover. It is possible by calculation to plot the general position of some of these concealed outcrops and to show the underground distribution of the productive and non-productive measures. A more detailed picture of the structure is given by contour maps of individual seams.

This East Pennine field is a good illustration of the shift of exploitation from the exposed to the adjacent concealed portion which is taking place in many other coalfields of this type. The known reserves in the Lancashire coalfield, for example, have been dwindling for many years with a decreasing annual output. Generally speaking, the main resources of the future are located along the southern margin of the South Lancashire and the eastern margin of the Cheshire fields where the Coal Measures plunge beneath the younger red rocks which extend across the Cheshire Plain. The National Coal Board in association with the Geological Survey has recently carried out a programme of deep boring which has disclosed workable coal at workable depth beneath these red rocks.

It is appropriate to mention here that large areas of eastern, central, and south-eastern England remote from the coalfields, are occupied by Mesozoic and Tertiary rocks which at widely varying depths rest upon a platform of older Palaeozoic rocks. The discovery of productive Coal Measures beneath the chalk near Dover more than 60 years ago led to the development of an entirely new coalfield in south-east England. It would be, perhaps, imprudent to forecast the discovery of other wholly concealed coalfields but it is part of the Geological Survey programme of boring, supported by geophysical investigation, to explore these older rocks at depth not only for obvious economic reasons but also to gain a fuller understanding of the geological history of this country.

Iron-ore.—Iron-ores are second in importance to coal in Britain, and annual production exceeds 10 million tons. Our reserves fall into two categories, the haematite ores, and the bedded or primary ores but the former yield less than 5 per cent of our total output. The principal bedded

ores are of Jurassic age and are found in eastern and central England. The bulk of the production comes from three levels but the Northamptonshire Sand ironstone at the base of the Inferior Oolite occupies much the largest area. The Survey has recently described this area and it may be taken as illustrative of its work in the iron-ore fields.

The Northamptonshire Sand ironstone occupies an area about 80 miles long and 20 miles broad which has been mapped in detail. The ironstone is generally persistent as a bed throughout the field but varies in character and in thickness from 12 to 20 feet, the workable portion being commonly 7 to 12 feet. The physical and chemical characteristics of the ironstone band have been studied in detail. The iron content averages 28 to 35 per cent and is the richest of our bedded iron-ores, all of which are of comparatively low grade.

The geological structure of the field is simple, the strata having a general easterly inclination of 30 to 40 feet a mile. There are locally small faults and occasionally small folds but practically everywhere the original structural pattern has been considerably modified by the development in comparatively recent geological times of structures of superficial origin, probably resulting, in the main, from the climatic conditions which prevailed during and after glaciation.

Most of the output of ore is at present obtained by opencast working and there is little underground mining. The character and thickness of rocks above the ironstone is evidently of great practical importance. The Survey has prepared maps showing the nature and thickness of the overburden. Considerable attention has also been given to estimating the reserves of iron-ore and the portion which is available for opencast working by present-day methods.

There are many other materials of economic importance in this country which I might discuss; for example, rock-salt, anhydrite, gypsum, potash, fluorspar, barytes, china-clay, the non-ferrous ores, and others. The work of the Survey in this general field of economic geology may be indicated by recording that apart from coal and iron-ore, it has published more than thirty Special Reports on the mineral resources of Great Britain and that many of them have been revised and brought up-to-date in second, third, and fourth editions.

The Investigation of Problems of Water Supply, particularly of Underground Water

The Survey has been concerned with problems of water supply for many years but during the 1939–1945 war, the demand for information necessitated an intensification of effort which resulted in the publication of about fifty Special Reports on underground water. Subsequently, and partly because of the Water Acts of 1945 and 1946, a separate department was set up wholly concerned with water supply problems but making use of the

fundamental geological information contained in the maps and memoirs of the Survey.

A primary well survey commenced during the 1939–1945 war has been extended over England and Wales and, since 1945, the volume of information collected has greatly increased because of the statutory obligation to notify the Survey of all new sinkings to a greater depth than 50 feet. The positions of wells have been accurately located on 6-inch maps with up-to-date information concerning water levels, yields, and so forth. Furthermore, these records include returns of the quantity of water pumped by large users who also measure the water levels in their wells each year; the effect of the withdrawal of known quantities of water can thus be watched. The task of gathering and interpreting this basic information has been considerable but researches are being carried out, as for example, on the transmission of water through rocks, on saline infiltration, and on the practicability of replenishing by artificial recharge water-bearing formation like the Chalk and the Triassic sandstones.

Owing to the need for information on the state of ground-water development in many areas, special surveys have been made to discover how much each water-bearing formation can yield and the quality it can produce. Surveys have been recently completed, for example, for the chalk of Yorkshire and north-east Lincolnshire in order to estimate the reserves of water available for future development. The hydrogeological map of north-east Lincolnshire shows the contour of the water-table, the areas of artesian flow, and of abnormal salinity whilst the amount of replenishment by rainfall and the loss by spring flow, and by pumping are estimated in order to reveal how much water is available for future development. In this particular instance, it was concluded that the area could produce a daily average of about 55 million gallons and, of that, about 7 million gallons are at present unused.

It is unnecessary to enlarge upon the contribution of the geologist in the choice of sites for reservoirs and dams in connexion with schemes for impounding water. However, mention may be made of the recently completed Bowland Forest tunnel on the Haweswater Aqueduct to Manchester, which passes through 10 miles of upland country south-east of Lancaster. A report by the Survey dealt with the nature of the rocks to be traversed, the structural arrangement of the strata, and its bearing on tunnelling, the geological stability of the area both near the surface and in depth, and the water-bearing characters of the ground. The Survey has investigated in the field and reported upon the proposals of the North of Scotland Hydro-Electric Board in relation to such items as catchment areas, dam sites, tunnel lines, surface aqueducts, and so forth.

An increased population, a rising standard of living, and the demand for industrial undertakings have meant an increased consumption of water. There is now the possibility of a greatly increased consumption in agriculture for summer irrigation both for crops and grass. Research has shown

that by supplying plants with all the water they require during growth, crops are much greater than they would have been if growth had been checked at intervals by moisture deficiency. There is a rainfall deficiency in 5 years out of 10 south-east of a line running roughly from the Humber to the Severn, the frequency of need increasing to 9 years out of 10 in the extreme south-east. It seems evident that irrigation can do much to increase production in south-east England. It is considered that very large quantities of water will be required to meet this potential demand. Estimates vary widely, but it appears possible that individual farmers may be able to use to advantage as much as 300,000 gallons daily for a period of 3 months for the irrigation of field crops on 150 acres, which is about the amount provided daily for a town of, say, 10,000 people.

The Discovery and Evaluation of Raw Materials, at Home and Overseas, for Atomic Energy

For some years, the Geological Survey has had a special department concerned with the discovery and evaluation of raw materials for atomic energy. Its activities are not confined to this country but also deal with investigations overseas in association with prospectors, mining companies, Dominion, and Colonial Geological Surveys. It gives advice on the value and possible development of radio-active ores, involving field work and the testing of materials by all available methods in the laboratory. Most of the investigations are concerned with the exploration and development of uranium prospects and mines. Such activities are, in large measure, based upon and advanced by research and, for that reason, constant attention is given to new developments in radiometric instrumentation, of new techniques of radio-assay and auto-radiography, and the perfection of more precise methods for the identification of radio-active minerals. In the exploration for ores, a great deal of work has been carried out on the production of light-weight ratemeters, with the development of borehole counters, on equipment mounted in vehicles for rapid reconnaissance over extensive areas, and in geochemical prospecting particularly in regions where the solid rocks are covered by a thick mantle of superficial deposits. There is no need to stress the importance of this work in relation to the production of atomic energy for power purposes.

The Maintenance and Development of the Museum of Practical Geology as a Centre of Instruction and Research

The Museum of Practical Geology contains a large amount of material on display and is open to the public. The exhibits are designed to illustrate the principles of geology, the regional geology of Great Britain, and the economic mineralogy not only of this country but also of the more important regions of the world. Guides to the collections have been published and

attention should perhaps be called particularly to those which deal with the regional geology of Great Britain. The attendance last year of more than 355,000 visitors to the Museum and nearly 6,000 persons to the lectures and demonstrations indicate the service by way of interest and instruction.

The reserve and study collections of rocks, minerals, and fossils are in constant use by the staff, particularly by the petrographical and palaeontological departments, but they are also available to accredited workers from this or any other country. The care and maintenance of these large collections, which include about a million specimens, devolve upon the staff of the Museum, who also undertake investigations of various kinds, for example, in mineralogy.

The Library which is open to the public for reference contains an extensive range of books, journals, and pamphlets on geology and ancillary subjects, and an especially representative collection of geological maps not only of the British Isles but of most countries in the world.

The Provision of Geological Information and Advice to Government Departments, National Boards, Industrial Concerns, and the General Public

The Geological Survey and Museum form a national repository of geological information for the service of Government departments and a wide range of other interested bodies. There continues to be an increased volume of enquiries and requests for special investigations; in recent years this has been due especially to legislation and Government policy in connexion, for example, with the public ownership of the coal-mining industry, with water supply, town planning, and so forth. Advisory work forms a large part of the activities of every section of the Survey and it is proper that this contribution should be made to the scientific and industrial life of the country but it is also important that it should not unduly impede the collection of new information and of adding to the scientific capital of the Survey in maps and memoirs.

I have not attempted in this lecture to deal directly with the interdependence of engineering and geology. I have been concerned, in accordance with the terms of your invitation, to give a general account of the history and work of the Geological Survey of Great Britain. The primary purpose of the Survey is to make geological maps and I have endeavoured to show the fundamental importance of such maps, particularly in certain aspects of economic geology, for example, in relation to coal, iron-ore, and water. It seems to me that this work is significant, in the words of your Charter of 1828, "for the general advancement of mechanical science and the art of directing the great sources of power in nature for the use and convenience of man."

Mr H. J. F. Gourley proposed and **Professor A. J. S. Pippard** seconded a vote of thanks to the Lecturer, which was carried with acclamation.

ELECTION OF ASSOCIATE MEMBERS

The Council, at their meetings on the 22nd June and 20th July, 1954, in accordance with By-law 14, declared that the undermentioned had been duly elected as Associate Members :

Home

Arnold, John Charles Thomas, B.Sc.(Eng.) (*London*).
Ashford, Reginald Frank.
Bak, Joseph, B.Sc.(Eng.) (*London*).
Ballard, Eric Hugh, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Barclay, Matthew.
Barker, John Arthur, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Barr, Stanley Herbert, Grad.I.C.E.
Beer, Alfred Edward.
Bloxsom, Alan Morris, B.Sc. (*Manchester*), Grad.I.C.E.
Braithwaite, John Capill, M.C., B.Eng. (*Sheffield*).
Brill, Herbert, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Brown, James Kerr, B.Sc. (*Glasgow*), Grad.I.C.E.
Buchanan, Robert William, B.Sc. (*Edinburgh*), Stud.I.C.E.
Bull, Peter Winslow.
Carlile, James Sim, B.Sc. (*Edinburgh*).
Caters, Harold Ezekiel, Grad.I.C.E.
Christodoulides, Socrates Panteles, B.Sc.(Eng.) (*London*).
Clark, Basil Edward George, Grad. I.C.E.
Clark, Sandy McAdam, B.Sc. (*Edinburgh*).
Clark, Thomas Edward, B.Sc. (*Wales*), Grad.I.C.E.
Clarke, John Aitken Gillespie, B.Sc. (*Edinburgh*).
Cockbain, Henry Michael Greig.
Collieson, Albert Fielding.
Connolly, John Raymond, B.Sc. (*Belfast*), Stud.I.C.E.
Coombs, David, B.Eng. (*Sheffield*), Grad.I.C.E.
Crocker, Trevor James Codrington, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Crockett, George Lumsden, B.Sc. (*St Andrews*), Grad.I.C.E.
Cross, Noel Victor.
Cruickshank, George, B.Sc. (*Glasgow*).
Curtis, George Ronald, B.Sc. (*Edinburgh*), Grad.I.C.E.
Curtis, Robert James, Grad.I.C.E.
Dabson, James Hugh, Stud.I.C.E.
Daly, Patrick Francis, B.E. (*National*), Grad.I.C.E.
Daniels, Robert James, B.E. (*Queensland*).
Davies, Peter Howell, B.Sc. (*Manchester*).
Day, Neville John, Grad.I.C.E.
Docherty, William Tysdale, B.Sc. (*Glasgow*).
Doran, Isaac Gregg, Ph.D., M.Sc. (*Belfast*).
Duncanson, Thomas Francis, B.Sc. (Eng.) (*London*).
Eastwood, Wilfred, Ph.D. (*Aberdeen*), B.Eng. (*Sheffield*).
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Emburey, Kenneth.
Evans, Clifford John, B.A. (*Cantab.*), Grad.I.C.E.
Farquharson, Ronald Campbell, B.Sc. (*St Andrews*).
Faulkner, John Brian Law, B.Sc. (*Birmingham*), Grad.I.C.E.
Ferguson, William, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Forster, Arthur Thomas Fletcher, B.Sc.Tech. (*Manchester*).
Gayner, John Francis Court, B.Sc. (Eng.) (*London*).
Glaze, Albert Gordon, B.Sc.Tech. (*Manchester*).
Glover, Hubert Alan, B.Eng. (*Liverpool*), Grad.I.C.E.
Goodchild, James Rupert, B.Sc.Tech. (*Manchester*).
Gorman, John, B.E. (*National*), Grad. I.C.E.
Graeme-Cook, Donovan, B.Sc. (*Belfast*).
Graney, Victor Thomas Charles, Grad.I.C.E.
Greenhalgh, Austin Kenneth, Stud. I.C.E.
Griffiths, William David.
Grundy, Walter, B.Sc. (*Manchester*).
Hale, John Andrew.
Halford, James Harold.
Helson, Peter William, B.Sc.(Eng.) (*London*).

HENDERSON, ROBERT MILNE, B.Sc. (*Edinburgh*).
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HOPPER, BRIAN BARNES, B.Eng. (*Liverpool*).
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KASIPPILLAI, KANDIAH.
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KELLAGHAN, PETER JOHN, B.E. (*National*).
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KEMP, KENNETH OLIVER, B.Sc.(Eng.) (*London*).
KENT, DAVID HARCOURT, M.A. (*Cantab.*).
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McKINNON, WILLIAM.
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McNISH, DENYS.
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SAUNDERS, JAMES EDWARD, Grad.I.C.E.
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STOCK, GUIDO, B.Sc. (*Edinburgh*).
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STRICKLAND, DAVID GARDNER.
SUNDERLAND, PATRICK HUGH, Grad. I.C.E.
SUTCLIFFE, RAYMOND, B.Sc. (*Manchester*).
SZEMBEK, ZYGMUNT JAN, B.Sc.(Eng.) (*London*).
TAYLOR, JOHN, B.A., B.A.I. (*Dublin*).
TAYLOR, PHILLIP AUSTIN, Grad.I.C.E.
THOMAS, ROGER MORGAN, B.Sc.(Eng.) (*London*).
THOMSITT, JOHN HENRY.
THORPE, WILLIAM ROBERTSON.
TYSALL, CYRIL ARTHUR.
VICKERS, DENNIS, Grad.I.C.E.
WADSWORTH, WILLIAM STANLEY ADIEL, B.Sc.(Eng.) (*London*), Grad.I.C.E.
WALKER, DENIS, B.Sc.(Eng.) (*London*).
WALKER, JOHN HALL, B.Sc. (*Birmingham*).
WALKER, KENNETH JOHN, B.Sc.(Eng.) (*London*).
WALLACE, ALEXANDER, B.Sc. (*Edinburgh*), Grad.I.C.E.
WALLER, JACK ARTHUR, B.Sc.(Eng.) (*London*), Grad.I.C.E.
WEEKS, PETER CHARLES, B.Sc.(Eng.) (*London*).
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WILLCOCKS, NORMAN VICTOR, B.Sc. (Eng.) (London), Grad.I.C.E.
WILSON, STANLEY DEAN, B.Sc. (*Leeds*).
WOODTHORPE, HERBERT JAMES, Stud. I.C.E.
WRIGHT, JOHN, B.Sc. (*Durham*).
YEO, JOHN, B.Sc. (*Bristol*).

Abroad

BADNALL, BEECHER FIENNES, B.Sc. (*Cape Town*).
BAKER, JOHN FREDERICK, B.E., B.Sc. (*New Zealand*).
BRADLEY, ALAN STEPHENSON, B.E., B.Sc. (*New Zealand*).
EARP, GEOFFREY NORMAN STOCKS, B.Eng. (*Sheffield*).
FABER, JOHN CECIL, B.A. (*Cantab.*), Grad.I.C.E.
JOLLY, ROY CECIL, B.Sc.(Eng.) (*London*), Stud.I.C.E.
KENNEDY, HENRY JAMES FORBES, B.E. (*New Zealand*).
MINETT, EDWARD PATRICK, B.Sc.(Eng.) (*London*).
O'SULLIVAN, JAMES BARNHAM, Stud. I.C.E.
RASAN, NAVARATNAM PUYAPALA.
SAVAGE, DONALD WILLIAM JOHN.
STEAD, HARRY DAVID, M.Sc.(Eng.) (*London*), Grad.I.C.E.
SUTHERLAND, WILLIAM MORLEY.
THOMPSON, NORMAN HAROLD, B.Sc. (*Nottingham*).
ZANDER, ISAAC DAVID, B.E. (*New Zealand*), Stud.I.C.E.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Robert Herbert Blackburn (E. 1909, T. 1945).
Col. Cyril Murton Croft, D.L., J.P. (E. 1935).
Francis Maurice Du-Plat-Taylor (E. 1919).
Albert Edward Leek, M.B.E. (E. 1913, T. 1938).
George McIldowie (E. 1913, T. 1936).
Alexander Murray (E. 1923, T. 1948).
John Amphlett Parker, M.C., B.Sc.(Eng.) (E. 1921, T. 1932).
Alexander Gibson Smith (E. 1910, T. 1925).
Smelter Joseph Young (E. 1910).

Associate Members

Eric Gaylmer Lytton Anderson, B.A., B.A.I. (E. 1929).
Francis Chaplin Holland, B.Sc. (E. 1920).
William Forster McMillan, B.Sc.(Eng.) (E. 1922).
James Herbert Mason, M.A. (E. 1914).
Sidney Gordon Basil Skingle, M.B.E. (E. 1938).
Eric Owen Stubbings, M.C., B.Sc.(Eng.) (E. 1915).
Percy Daniel Whitestone, M.A. (E. 1949).

Associate

Edward Fulton Law (E. 1936).

CORRESPONDENCE

on a Paper published in Proceedings, Part I, January 1954

Paper No. 5939

"The Elastic Stability of Straight I-beams subjected to Complex Loads"†

by

Philip Michael Worthington, B.Sc.(Eng.), Grad.I.C.E.

Correspondence

Mr A. N. Procter commended the Author's attempt to reduce some of the large mass of buckling theory to a form in which it might be applied to design, so that engineers could judge to what extent such theories might be used in practice. The Paper contained a number of ideas and methods which helped to clarify and simplify the general theory, but Mr Procter was in some doubt as to the experimental accuracy of some of the basic formulae which were introduced at the beginning of the Paper; that applied particularly to those referring to torsion (formula (7)), to torsional buckling (formulae (5) and (6c)), and also to bending moments applied normal to the web (formulae (19) and (20)).

There was little doubt that a piece of wire under torsion could become unstable, but Mr Procter did not know whether Professor A. G. Greenhill's formula (footnote reference, p. 50) had received conclusive experimental confirmation for practical sections. For a solid round bar that formula, in terms of the Author's symbols, would become: $T_c = \pm \frac{2\pi\beta}{l}$. In terms of the calculated maximum shear stress, that became: $f_{sc} = \pi E \cdot \frac{k}{l}$ and it was found that with a slenderness ratio of $\frac{l}{k} = 400$, the calculated maximum shear stress f_{sc} would exceed 100 tons per square inch, or alternatively, if the maximum shear stress was limited to 6·5 tons per square inch, elastic instability would not occur until the slenderness ratio was well over $\frac{l}{k} = 6{,}000$, so that, for practical cases, that kind of buckling might not be

† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 46 (Jan. 1954).

of very much importance,[1] and Professor Greenhill's formula might give results which were too high. Therefore, the Author's adaptation of that formula to I-beams (see p. 62) would also be expected to give results on the high side. For a 24-inch-by-$7\frac{1}{2}$-inch 95 lb. per foot R.S.J. limited to 50 feet long $\left(\frac{l}{k}=400\right)$, with restrained ends, the buckling torque would be 4,300 foot-tons; that would give a very high maximum shear stress and, for a limiting shear stress of 6·5 tons per square inch, the beam would have to be several miles long before instability occurred.

On the other hand, for torsional buckling under axial loading, formula (6c) appeared to give results which were rather the opposite. Putting $\gamma = N \,.\, J$, where J was the torsion constant $(\frac{1}{3}Dt_w^3 + \frac{2}{3}Bt_t^3)$, formula (6c) reduced to: $P_{sc} = \frac{1}{r^2}\left(N \,.\, J - \frac{4\pi^2 E \,.\, I}{l^2} \times \frac{D^2}{4}\right)$ so that the critical stress with ends restrained was given by: $p_{sc} = \frac{D^2}{r^2}\left(\frac{N . J}{D^2 . A} - \pi^2 E\left(\frac{k}{l}\right)^2\right)$. Using, as before, a 24-inch-by-$7\frac{1}{2}$-inch 95 lb. per foot R.S.J. it was found that p_{sc} would be zero for all values of $\frac{l}{k}$ up to 260. Referring, however, to the original formula (5), which was attributed to H. Wagner, it appeared that γ and $\frac{P}{r^2}$, bracketed together, were dimensionally dissimilar, so that the theoretical authenticity of formula (5) was doubtful.

Professor Lea considered that buckling of duralumin struts of various sections during failure was caused by "the lips buckling and the strut apparently twisting," [2] and that also appeared to be implied by Professor S. Timoshenko, so that it appeared that an I-beam of normal proportions would not buckle in that way while in the elastic state, but it might do so when there was local yielding of the material due to ductility.

In the matter of the lateral bending moments (M''), given by formulae (19) and (20), it was noted by the Author that those values were always imaginary, but it might not follow from that that a lateral bending moment, applied normal to the web of an axially or transversely loaded beam, would increase its stability, since it had been established experimentally on several occasions that such bending moments applied normal to the web of axially loaded joists considerably reduced their stability,[3, 4, 5] in the same way that they also reduced the stability of round-section struts.[6, 7] It appeared, therefore, that Mitchell's formula might, in fact, be used only for slender beams bent about the major axis and that a different treatment was required for the bending moments M''.

Referring generally to the question of elastic buckling under complex loading, it appeared necessary not only to provide means of estimating the buckling conditions, but to provide also a means of determining the secondary stresses, since it might often happen that a flexible member

under a complex system of loading such as was being considered might fail due to overstressing of the material before the theoretical buckling loading was reached.

The Author, in reply, agreed that the torsional stability of steel I-beams of the proportions usually found in structural buildings was not important. The theory of torsional stability had been included in the Paper for the sake of mathematical completeness, so that all possible loads that could be applied to the beam had been considered. One example of the application of the theory was in the design of a suspension for a moving coil galvanometer. Another example was in the design of shafts and torsion bars that were unsupported over considerable lengths. Instability under torsion and tension combined was seen when the knots first appeared in the elastic engine of a model aeroplane.

The Author regretted that formula (5), taken from Goodier's Paper,[8] had been incorrectly quoted. The formula should have been written:

$$T = (\gamma - P_r^2)\frac{d\theta}{dx} - a\frac{d^3\theta}{dx^3} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

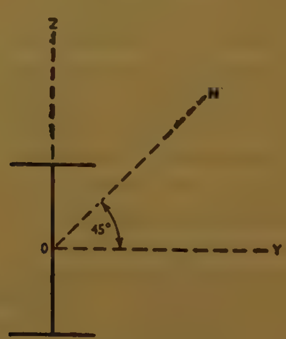
There was also an error in equation (7) which should have been written:

$$T_c = \pm \frac{t}{l}\sqrt{\beta_1\beta_2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

The negative sign in equation (6*c*) should be positive. Those errors did not effect the remainder of the Paper.

Throughout the Paper, the Author had only treated the stability of the beam as a whole. The local buckling of flanges or web, although very

Fig. 3



important in many cases, was not dealt with in the Paper. The failure by twisting caused by an axial load was a case of buckling as a whole and not local buckling or yield. Unless an I-beam was constrained in some unusual way it failed by bending about the weak axis before failure by

twisting could occur. That was because the value of $P_{\theta c}$ given by equation (6*c*) was always greater than that of P_{yc} given by equation (6*a*). Nevertheless, in complex cases, the value of $P_{\theta c}$ needed to be computed for substitution in formula (42).

The Author considered that Mr Procter was confusing deflexion and stress with the conception of the " critical magnitude " of a load system (defined on p. 53). The calculations in the Paper referred only to the critical magnitude assuming that the beam remained within the elastic range. Whilst a system of lateral bending moments, M'', increased the deflexions and stresses in a beam it also increased the value of the critical magnitude of the load system. If a beam was subjected to M' and M'' only, M'' being equal to or greater than M', it was seen from equation (31) that there was no real magnitude of the load system which put the beam in neutral equilibrium. Thus a resultant load acting in the sector ZON in *Fig. 3* had a real critical magnitude, whilst a resultant load acting in the sector NOY only had an imaginary critical magnitude.

References

1. Arthur Morley, " Strength of Materials." Longmans Green & Co., 1908.
2. " Report on Materials of Construction used in Aircraft," p. 70. H.M.S.O., 1920.
3. Von M. Ros, " *Die Bemessung zentrisch und exzentrisch gedrückter Stäbe auf Knickung* " (" Study of bending failure in axially and eccentrically loaded compression members "). 2nd Int. Cong. Br. Struct. Engrs, Vienna, 1928, p. 282.
4. Julius Brummer, " *Neue Methode der Aufstellung hoher Eisenfachwerksäulen und Maste mittels Doppelhebel* " (" A new method for the erection of high steel structures and masts by means of double levers "). *Loc. cit.*, p. 614.
5. B. G. Johnston and L. Cheney, " Steel Columns of Rolled Wide Flange Section." Ctee on Tech. Res., Am. Inst. Steel Constr., Progress Report 2, 1942.
6. W. E. Lilly, " Eccentrically-Loaded Columns." Min. Proc. Instn Civ. Engrs, vol. 181 (1909–10, Pt III), p. 460.
7. Andrew Robertson, " The Strength of Struts." Selected Engineering Paper No. 28, Instn Civ. Engrs, 1925.
8. S. Goodier, " Torsional and Flexural Buckling of Bars of Thin Walled Open Section Under Compressive and Bending Loads." J. Appl. Mech., U.S.A., 1942.

CORRESPONDENCE

on a Paper published in Proceedings, Part I, January 1954

Paper No. 5959

"A Foundation Failure due to Clay Shrinkage caused by Poplar Trees"†

by

Alec Westley Skempton, D.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr W. H. Elgar observed that the first case in which the principle of responsibility for damage to buildings by water-absorbing trees had been admitted in law was that of Butler *versus* The Standard Telephone & Cable Company.[18]

Damage to buildings by poplar and other trees growing on clay subsoils was a very common occurrence in the Cambridge district. Damage occurred most frequently where the subsoil was Gault and rather less frequently on the boulder clay.

The first case of that type which Mr Elgar had investigated occurred in 1941. Since then he had had to deal with similar damage to a large number of buildings including private houses, farm buildings, and University laboratories. At the present time he was carrying out underpinning work to support a two-storey laboratory, where serious damage had occurred owing to the proximity of poplar trees. The soil consisted of gravel, from 4 to 6 feet deep, overlying Gault clay.

It was sometimes supposed that the shrinkage of clay caused by a tree was confined to the volume of soil which was actually occupied by the tree roots, but that was not the case. The reduction in moisture content took place in a zone larger in area and considerably deeper than the limits of penetration of the tree roots. Investigations which Mr Elgar had undertaken following the drought of 1947 had shown that the soil beneath well-grown poplar trees, 60 or 70 feet high, had been affected to a depth of 15 or 16 feet and that the maximum reduction in moisture content had occurred at depths from 7 to 9 feet below the surface.

Experience seemed to show that, in the case of light buildings, more damage was caused by the expansion of the clay subsequent to a period

† Proc. Instn Civil Engrs, Part I, vol. 3, p. 66 (Jan. 1954).

18 For reference 18 *et seq.*, see p. 621.

of drought than by shrinkage during a drought. Heavier buildings exerted sufficient downward pressure to counteract the uplift of the expanding subsoil.

The behaviour of tree roots was often extraordinary. A few years ago part of the retaining wall supporting the grounds of Magdalene College, Cambridge, had been severely damaged by the roots of a number of very fine horse chestnut trees, which had penetrated the wall and were causing it to collapse gradually into the River Cam. A root from one tree was found to have grown upwards out of the ground into the parapet wall and to have penetrated through it longitudinally for a distance of over 40 feet, sending out numerous smaller roots and lifting the coping stones bodily. An explanation offered by a botanist was that the soil was generally deficient in lime and the tree accordingly sought to remedy that deficiency by sending out a root to obtain it from the lime mortar of which the wall had been built. The retaining wall had since been rebuilt under Mr Elgar's direction and the trees removed.

About 2 years ago Mr Elgar had examined the root of a large elm tree which had grown just below ground level for a distance of about 150 feet from the base of the tree and, in so doing, had passed through the soil-filling over an arched bridge to penetrate the soil on the side remote from the tree.

Mr Elgar was in complete agreement with the provisional rule suggested by the Building Research Station that the "safe distance" of a tree from a building on clay subsoil was equal to the height of the tree, but his own experience did not support Mr Skempton's suggestion that a single tree might with safety be rather nearer. The effect of a single tree in proximity to a building, particularly if it was near a corner, often seemed to be more serious than that of a row of trees owing to the increased differential settlement between adjoining parts of the building.

Mr Elgar had used short-bored piles for the foundations of buildings on clay subsoils for some years, but not always in the precise form recommended by the Building Research Station, and he had also used a modified arrangement for underpinning buildings which had been damaged by settlement. So far he had not experienced any case in which settlement had occurred where short-bored pile foundations had been used for buildings of one or two storeys.

Mr D. A. Harris commented that, although the Author had pleaded for the publication of further case records of similar troubles due to clay shrinkage, his (the Author's) Paper about a case which had been investigated and dealt with in 1947 did not appear until 1954. As the Author had stated, the problem concerned both the architect and the engineer, and thus emphasized the need for close contact between the two professions.

Regarding the case described in the Paper, although the extent of the underpinning carried out was not stated, it appeared likely from the

site plan that about 60 feet of underpinning might have been necessary. Had any consideration been given to a possibly cheaper remedy, namely, to dig a trench about 10 feet deep (well below the lowest roots) just inside the boundary wall, cutting off and killing all the roots which had trespassed and caused the damage by clay shrinkage, and placing in the trench before back-filling a shield of, say, sheet-piling, to prevent a second encroachment by the roots ?

Such a shield might, to judge from the site plan, have had to be 70 feet long, but might well have been easier and cheaper to provide, whilst protecting not only the theatre but any other buildings that might be erected in the future between the theatre and the boundary.

Mr J. H. G. King and **Mr D. A. Creswell** said that the lack of information about the effects of trees on foundation subsoil mentioned by the Author prompted the following brief analysis of three foundation failures. It was regretted that the examinations were not so complete or exact as those given by the Author, but it was hoped that the facts reported would amplify some of his findings and recommendations.

Case I.—A large detached three-storey brick-built house about 60 years old in Hornsey, North London. The foundations seemed adequate, and were about 4 feet below ground level in London Clay (see *Fig. 6*).

The property was first examined in the autumn of 1951, when it was found that the front of the building and part of the return wall (No. 1) had apparently settled about 1 inch. Walls Nos 2 and 3 had cracked in many places and were bonded into the front and rear walls of the building.

The line of trees, closely planted, consisted of Lombardy poplars and chestnuts, approximately 55 feet high. All the surrounding ground was paved and drained to sewers, but the impervious paving of the forecourt was thought to have been laid recently, probably since 1945.

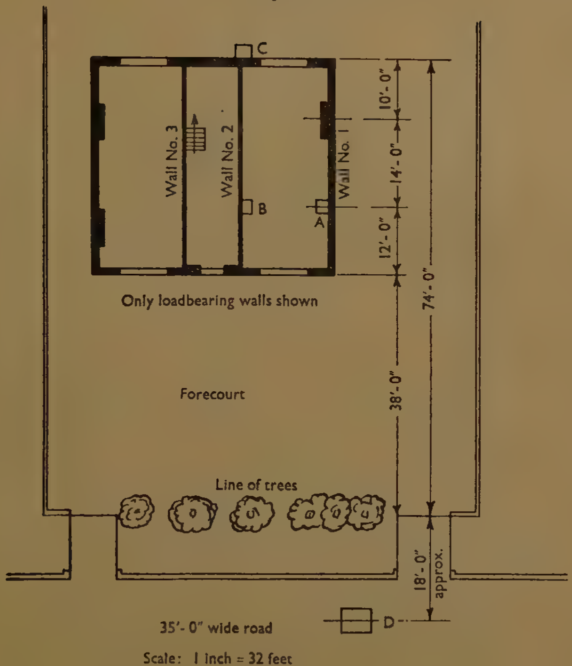
Trial holes were excavated at points A, B, and C. The foundation subsoil was brown clay with pockets of grey clay and the shear strength averaged 1,010 lb. per square foot at the underside of the foundations.

Tree roots were found in both holes A and B, one root $1\frac{1}{2}$ inch in diameter passed below the footing at A ; later, during the underpinning process, roots nearly 1 inch in diameter were discovered below the chimney in wall No. 1. That would indicate that the roots must have penetrated beyond the back wall—a distance of 74 feet from the line of trees. It was thought that the trees had been planted after the house was built but that was not certain. There were no trees near the rear or sides of the building.

Since the brickwork was bonded with lime mortar there were no large external cracks, but there were many internal ones in the plaster. The rear wall was nearly 2 inches out of vertical, leaning in at the top. It might have been built in that way, but the more likely explanation was that it had pulled over when the front settled.

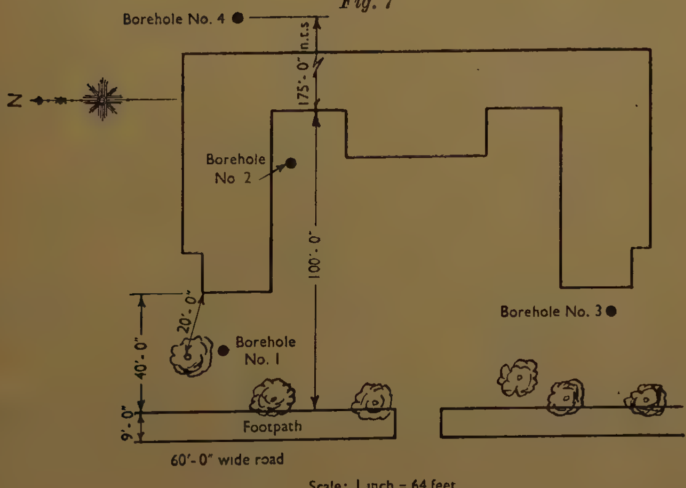
During the autumn of 1952, when the underpinning was taking place,

Fig. 6



CASE I

Fig. 7



CASE II

the Local Authority opened the road at point D to inspect a drain. Tree roots were found around the joints in the drain, 14 feet below the road level. The major points of interest about that examination were the distance and depth to which the roots had travelled in their search for water, once the natural catchment area had been restricted.

Case II.—A large two-storey brick building erected in 1932, near Harrow (see *Fig. 7*). The pitched roof was tiled and the floors were 5-inch-thick reinforced-concrete slabs. The foundations were of adequate size, and their depth below ground level varied from 2 feet 6 inches on the north wing to 3 feet 6 inches on the south wing. Those foundations were in fact level, but the ground sloped down from north to south. In an east-west direction the site was flat.

The first signs of movement were that the gables and side walls of the wings developed cracks in 1938. By 1947 the damage was widespread throughout both wings, and it was decided to fix raking shores to the gables. A particularly significant crack at the end of both wings seemed to indicate that they had both broken away from the main building.

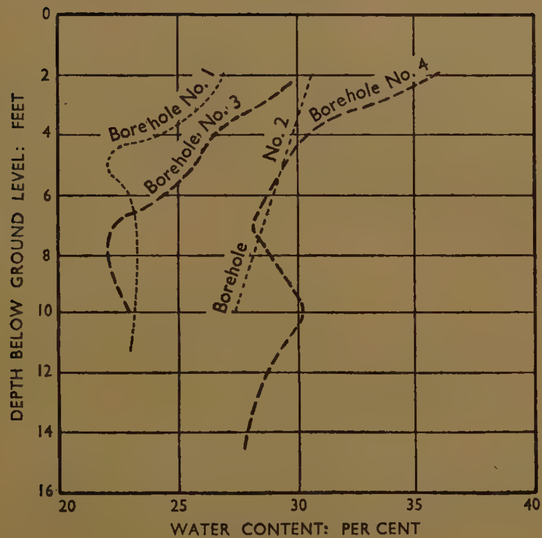
In January 1948 a soil survey was made, and boreholes were put down in the positions indicated in *Fig. 7*. Below the usual top soil, brown clay was found to a depth of 15 feet, with blue clay beyond. The shear strength of undisturbed samples ranged from 800 to 1,000 lb. per square foot. Some traces of sulphur trioxide were found at foundation level and a few large crystals of gypsum were extracted from a depth of 10 feet below ground level. When the foundations had been exposed, however, they were all found to be in perfect condition with no signs of any sulphate attack.

The variation of water content with depth of subsoil is shown for each of the four boreholes in *Fig. 8*. The result for borehole No. 4, which was in open ground well to the rear of the buildings, might be regarded as the normal condition of the soil. The sample from borehole No. 2 suggested that the water content had been affected near ground level by the large number of shrubs. The results from boreholes Nos 1 and 3 showed that the clay had dried out considerably to a depth of 10 feet. That was fairly conclusive evidence that the oak trees, all of which were about 50 feet high, and some only 35 feet from the nearest part of the building, were responsible for the damage resulting from settlement.

Case III.—A single-storey brick building, erected in 1936 at Edgware. The tiled roof was supported by steel trusses. There was a small basement, which accommodated the boilers, at a depth of 10 feet below the ground slab (see *Fig. 9*).

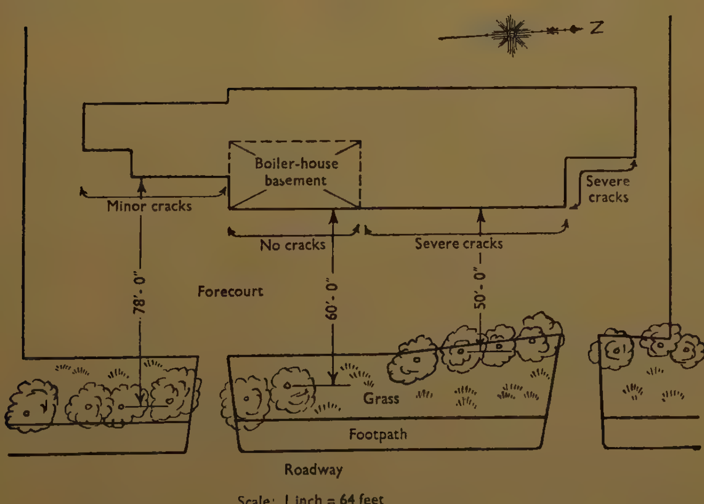
The large forecourt was paved with tarmacadam, and the surface water was carried to the main drainage system. The only open ground which could act as a catchment area for the trees was a grass verge between the site boundary and the public footpath. The elm trees in that verge were 50 to 60-feet high.

Fig. 8



WATER CONTENTS OF SOIL SAMPLES

Fig. 9



CASE III

Settlement cracks on the front elevation developed over a number of years; in 1949 they were considered to be serious and a detailed examination was carried out. The cracks were more extensive at the north end of the building, which was nearest to the trees. There were no cracks in the part of the building over the basement boiler-house.

The foundations were exposed at three places on the east wall. The ground was brown clay, and the underside of the footings was 3 feet 6 inches below ground level; it was considered that they were of sufficient size to distribute the load adequately.

No soil strength tests were carried out, but a chemical analysis of the soil and ground-water showed :—

Ground-water .	. 320 parts per 100,000 of sulphur trioxide.
Soil . . .	. 0·78 per cent sulphurous anhydride.

The mortar below damp-proof course was dark brown in colour and it crumbled very easily between the fingers. Mortar above the damp-proof course was in a perfectly sound condition.

The concrete in the foundations was rather porous and the voids were filled with a white powder, which, when analysed, proved to be hydrated calcium sulpho-aluminate. The concrete was probably made with alumina cement in a 1 : 3 : 6 mix. There was evidence of hydrolysis due to water passing through the voids, and that sulphate had been taken up by the lime set free. There was no evidence of normal sulphate attack.

Tree roots were found around and below the foundations, particularly at the northern end of the building.

It was difficult to say which was the principal factor causing settlement in that case, the trees or the unsatisfactory concrete. In the restoration work, dense 1 : 2 : 4 high-alumina cement concrete was used in the foundations and underpinning works, and those had been satisfactory.

Mr T. K. Chaplin commented on the "safe distances" suggested on p. 81 under the heading of practical recommendations. Despite the reference to Mr W. H. Ward's Paper on "Soil Movement and Weather," the Author's conclusions were not supported by the detailed information given in that Paper.

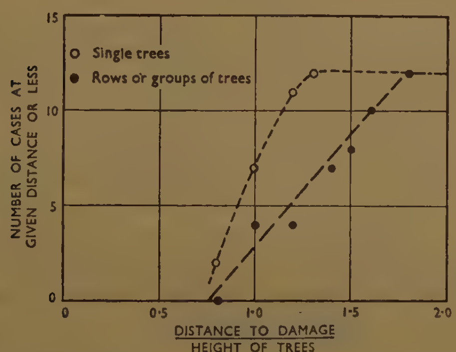
Mr Ward's results included twenty-four observations of actual damage, twelve by single trees and twelve by rows and groups of trees. Mr Chaplin had plotted the cumulative distribution of the ratios of distance to damage divided by height of tree (see *Fig. 10*). It appeared that a single tree could cause damage at a distance of up to about $1\frac{1}{4}$ times the height of the tree, and a row or group of trees up to about $1\frac{3}{4}$ times the height of the trees. He could not help feeling that it was wiser to be on the safe side and use those maximum distances rather than use smaller distances based on two isolated individual cases where cracking had not been observed.

Curiously enough, although cracking would not be expected above the deep basement of the cinema, *Fig. 1* (facing p. 72) appeared to show

cracking of the main wall above the left-hand edge of the air inlet, which was 11 feet from the edge of the basement. From *Fig. 2 (a)* (p. 69) that crack was about 55 feet from the trees, or at a distance of 1·3 times the tree height.

Referring to the Appendices, Mr Chaplin pointed out that the plastic flow of a cohesive layer and the bearing capacity of a footing on such a layer were two arbitrary aspects of the same problem. To analyse the two aspects separately might be seriously on the unsafe side. The Author had quoted factors of safety against plastic flow and bearing capacity failure of 3½ and 4·2 respectively, whereas an analysis by Mr Chaplin for the combined problem gave a factor of safety of less than 3.

Fig. 10



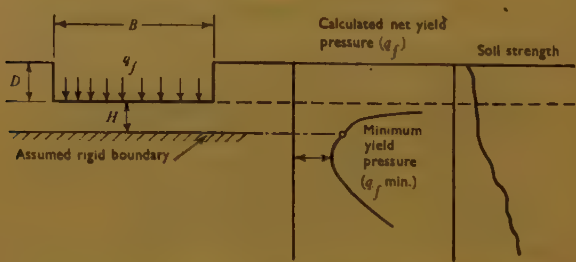
DISTRIBUTION OF DISTANCES BETWEEN TREES AND ASSOCIATED DAMAGE

The complete analysis of footings on cohesive layers had been given by Dr Meyerhof and Mr Chaplin.[19] The analysis in that Paper had been based on a material of uniform strength; it could, however, be easily extended to non-uniform material by considering the adhesion component separately, but each problem had to be worked out several times until the minimum yield stress had been found.

In *Fig. 11* for any position of an imaginary rigid boundary at depth H there was a corresponding yield pressure q_f (bearing capacity). That had a minimum somewhere, depending on the variation of strength with depth. To apply the method to the problem in question Mr Chaplin had used a smooth curve of strength at depth, based on the Author's quoted strengths, for the purpose of discussion. The calculations gave a minimum yield pressure of 6,940 lb. per square foot, equivalent to a factor of safety of 2·9. The Author's analysis, considering the aspect of possible plastic flow of a localized soft layer 2 inches thick with a strength of 400 lb. per square foot, gave a yield pressure of 8,580 lb. per square foot, equivalent to a factor of safety of 3·6.

The difference between the two methods of analysis was about 23 per cent, but that was only a very approximate figure owing to the lack of information about the soil strengths immediately under the footing. However, it was clear that the lower factor of safety was still so far above unity that the likelihood of a failure by plastic flow or of a bearing-capacity failure was non-existent. The main purpose in describing the comprehensive analysis was to demonstrate that the calculation of bearing capacity of foundations should take into account the continuous variations of soil strength in order to find the minimum value. It was rather like a slip circle analysis which had to be repeated many times, assuming different failure surfaces, to find the factor of safety of a slope.

Fig. 11



CALCULATION OF MINIMUM YIELD PRESSURE—GENERAL CASE (PURELY DIAGRAMMATIC)

TABLE 3

$\frac{c_1}{c_0}$	$\frac{q_f}{c_0} = N_c$ Strip foundation	Circular foundation
1·0	5·1	6·2
1·2	5·5	6·4
1·4	5·8	6·6
1·7	6·2	6·9
2·0	6·5	7·1
2·5	7·0	7·4
3·0	7·3	7·6
4·0	8·0	8·0
6·0	9·1	8·9

c_1 denoted soil strength at depth below footing equal to footing width (or diameter).
c_0 ,, soil strength at surface.
q_f ,, bearing capacity.
N_c ,, the corresponding bearing capacity factor.

Since the case where soil strength increased linearly with depth was not uncommon, Mr Chaplin had also calculated the bearing capacity of a surface footing on such a soil. The results showed (see Table 3), that application of the simple theory of bearing capacity using the mean strength down to a depth of B, or $\frac{2}{3}B$, would be on the unsafe side.

Finally, Mr Chaplin said he entirely agreed with the Author that it was the poplar trees which had caused the damage to the cinema, and not some extraneous influence such as plastic flow.

The widespread damage caused by tree roots was still an extremely serious problem, and he thanked the Author for drawing attention to it by his Paper and for laying the blame for the damage so squarely on the tree roots.

Mr R. F. Legget, of Ottawa, Canada, observed that the Paper would be of particular interest to many Canadian engineers and builders, since in some regions of Canada great damage had developed to buildings founded on clays which were subject to shrinkage and swelling movements. Those movements were not always associated with the drying effect of trees, but frequently the magnitude of damage appeared to be greatly influenced by the proximity of trees.

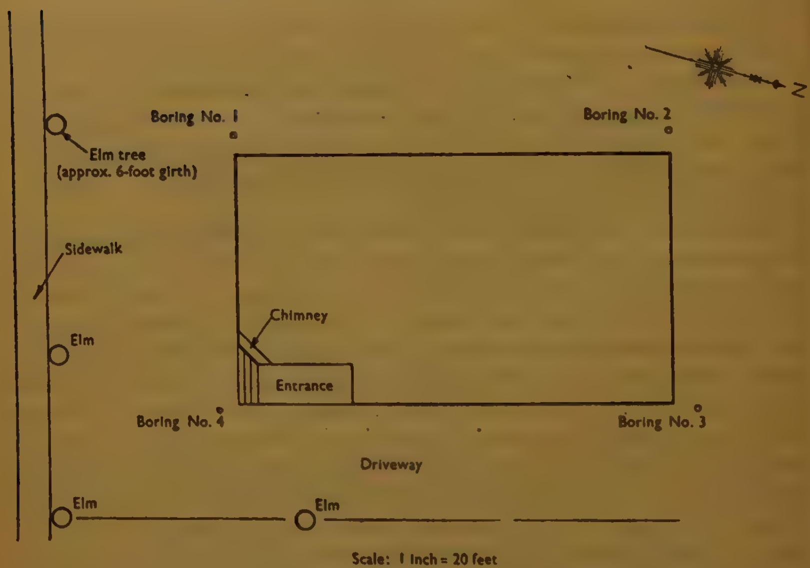
In general, the highly plastic gumbo clays of Western Canada were responsible for the greatest foundation movements but the problem had been observed to varying extents from coast to coast. A detailed investigation of the foundation movements of the Regina College gymnasium had been reported by Professor B. B. Torchinsky.[20] That two-storey brick building was supported by spread footings at a depth of 5 feet, and it was estimated that the outer walls had settled about 8 inches. Although the average swelling pressure of clays in the Winnipeg region was considerably less than that found farther west at Regina, significant ground movements had been observed where foundation loads were small. Much damage had been attributed to that cause during the Red River flood of 1950. Lightly loaded interior footings were lifted as much as 3 inches and most distortion of basement floors was attributed to that cause.[21] Direct observations of ground movements at Winnipeg were begun in 1951. Details of those studies were reported in 1952 by Professors A. Baracos and O. Marantz[22] of the University of Manitoba.

In some regions of Canada seasonal ground movements would extend to a considerable depth even without the added drying effect of trees. In Ottawa, however, there was evidence to show that most of that type of damage was related to the growth of trees. It was common to find undulating kerbs and sidewalks. The depressions in most cases appeared to be directly related to the location of large trees.

One excellent example, in Ottawa, of structural damage caused by subsoil drying was investigated in September 1948. The structure, a two-storey double house of brick construction with a concrete semi-basement, was built in 1946. *Fig. 12* showed a plan of the site together with

the location of four auger borings which were made in connexion with the investigation. The borings revealed brown sand to a depth of 5 feet, overlying a highly plastic clay with a liquid limit of about 79, a plastic limit of about 27, and a plasticity index of 52. That clay graded into a soft, sticky, grey clay at a depth of 10 or 12 feet. Soil conditions appeared to be fairly uniform except at boring No. 4 where the clay was stiffer and very dry near the sand-clay interface. Natural water contents ranged from 60 to 75 per cent at borings Nos 1, 2, and 3 but decreased to about 50 per cent at boring No. 4.

Fig. 12

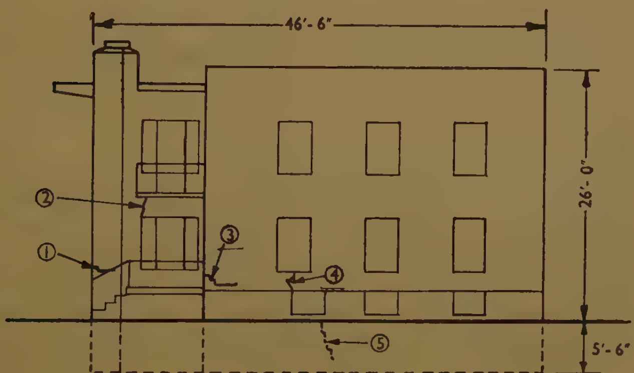


SITE PLAN OF OTTAWA BUILDING

The soil investigation was made following a report that the south-east corner of the building had suddenly experienced severe cracking and that doors and windows were jamming. Several serious cracks were observed which indicated a local settlement of that corner. Some of the cracks had opened to a width of about 1 inch. It was also observed that the sidewalk in front of the building had undergone a remarkable downward displacement. Discussion with the owner revealed that the cracking had greatly increased during August and September. Both families, resident in the building, had been absent during most of the summer and the lawn had not been watered since early summer. Rain water was collected on the roof and discharged into the sewer.

In view of the relatively light loads and the two-year interval between construction and the appearance of damage, it seemed certain that the cause of settlement was not due to excessive loading on the soil. That opinion was further strengthened by the fact that boring No. 4, at the damaged corner, revealed a harder clay than existed at the other borings. It was clear that local drying of the clay had been responsible for the damage and that it had probably been caused by the proximity of large elm trees (shown in *Fig. 12*), the trees being about 35 feet high. The condition of the sidewalk gave further support to that reasoning. A study of "evapo-transpiration" values showed that conditions were about average for Ottawa during the summer of 1948. The usual period

Fig. 13



EAST ELEVATION OF OTTAWA BUILDING

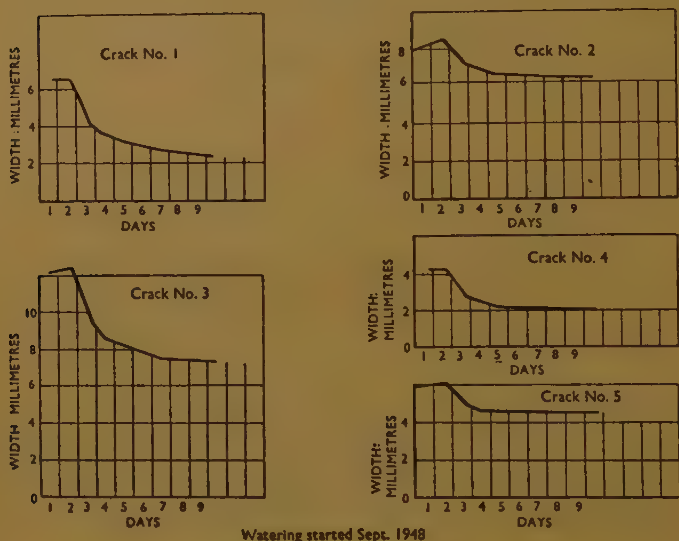
of a slight soil moisture deficiency was observed, beginning early in August and lasting until the end of September.

It was decided that an immediate treatment to arrest further movement of the building would be to supply water to the soil artificially and it was suggested that the adjacent elm trees should be removed. Subsequently, the tree directly east and the one directly south of the damaged corner were cut down. After making careful measurements of the crack dimensions, the lawn was watered for 2 or 3 hours on the 28th September. On the following day the garden hose was placed under the entrance step and watering was carried out almost continuously until October 7th. The location of the cracks and the effect of artificial watering on them was shown in *Figs 13* and *14*. It would be seen that the horizontal cracks closed by about 50 per cent after about 5 or 6 days and that the vertical cracks closed about 30 per cent after 4 days. At the last inspection of

that building, in May 1954, the cracks had been repaired and were barely noticeable. No further damage appeared to have occurred. At that time, a good deal of amusement had been caused by the recommendation by research workers that the lawn should be watered, but the results which were obtained almost immediately completely justified their suggestion.

In view of the extent and severity of that type of damage to light

Fig. 14



WIDTH OF CRACKS PLOTTED AGAINST TIME FROM START OF WATERING

structures in the Ottawa area, the Division of Building Research of the National Research Council was preparing to make a detailed study of the problem. An attempt would be made to determine the cause of damage to many buildings, to evaluate the degree of damage, and to relate those facts to the soil types. From that study it was hoped to produce suitable design criteria for foundations of light structures for soil conditions such as those of the Ottawa area.

The Author, in reply, stated that the phenomenon of shrinkage of clays, caused by tree roots, led to two types of problem in practice. First, the choice of procedure to be followed in the construction of new buildings, and secondly, the choice of remedial methods to be used where an existing building had suffered damage. In the case of new construction the architect should be free to place the building where he wished, in relation to existing trees, or to plant trees where he wished, independently of the soil conditions. But that freedom carried with it the responsibility

of assessing the likelihood of cracking arising from the drying action of tree roots and, if necessary, of designing a foundation that would eliminate that danger. Similarly, it was the responsibility of the research worker to provide date from which safe rules could be established for the guidance of the architect.

Present-day knowledge indicated that if a building was to be founded on clay and if it was to be placed at a distance of less than H feet from a deciduous tree, where H denoted the full-grown height of the tree, then (*a*) the building should be on piles (or piers) or provided with a basement, and (*b*) the piles or the footings should obtain their bearing in soil below the summer ground-water level or below the greatest depth of root penetration, whichever was the smaller. More specifically, the " safe distance " should be expressed as $x . H$ where x was a multiplying factor dependent upon the species of tree and site conditions; the available evidence suggested that, as a rough rule, $x = 1$ for single trees of such species as poplars, elms, oaks, and limes. If the space between the trees and the building was paved with impervious concrete or macadam, the factor x was likely to be greater than unity and if there was a group or row of trees then, again, the factor x should be increased, probably to 1·5. On the other hand, the Author was still of the opinion that the value of x might be less than unity for a number of deciduous species, since the water requirements of all trees of a given height could not be constant, nor was it likely that the root spread was the same. In that connexion the predominance of poplars, elms, and oaks in the reported cases of damage was remarkable; although, as Mr Ward had pointed out, oaks and elms were among the dominant species in the clay lands of south-eastern England, and poplars the most frequently planted trees in the suburbs. In contrast, few cases of damage were yet known to result from evergreens, and in those cases the trees were very close to the buildings.

For remedial measures one possibility was to cut down the trees and make good the cracking in the building. Mr Legget had given a very interesting example of that procedure, in Ottawa, and the Author knew of other examples in the London area where the removal of trees had brought about a permanent remedy. But in other cases it might be very undesirable to eliminate the trees; it was then necessary to underpin the building, with deep footings or piles, to obtain a bearing on ground beneath the zone of shrinkage. The Author knew of a building which had cracked badly as a result of shrinkage caused by a row of magnificent oak trees and, in that instance, the decision was rightly made to preserve the trees and underpin the foundations with piles about 15 feet long.

With regard to individual points mentioned in the correspondence, Mr Harris had raised the question of a shield trench. That had been considered at Stamford Hill and, in principle, it was probably a sound expedient, provided that the trench was taken deep enough to prevent the roots from penetrating underneath and provided that the trees were not killed or

rendered unstable in high winds by the cutting off of their roots. But the theatre wall foundations required strengthening and it was decided to achieve that by underpinning, and to take the footings to a depth sufficient to eliminate danger from the roots. Another possibility was to construct a gravel-filled trench of moderate depth and provide a source of water to that trench in order to irrigate the clay.[23] That solution appeared, however, to be rather hazardous owing to the difficulty of ensuring a supply of water during long spells of dry weather, when the water was most needed.

Mr Chaplin had referred to a graph published in 1953 by Mr Ward which indicated that a single tree could cause damage at a distince up to about 1·25 H, and a row or group of trees up to about 1·75 H. Those distances were, perhaps rather exceptional. But in view of Mr Ward's graph and additional data referred to later, the Author would modify his earlier views to the extent that for a row of poplars, elms, or oaks, the safe distance should be considered as 1·5 H, especially if the ground between the trees and the building was paved. For a single tree of those species, and with unpaved ground, the safe distance appeared to be equal to the tree height. Since those species grew commonly to a height of 50 or 70 feet it would be seen that the recommendation in Clause 2.221 of the Code of Practice on Foundations (1954) that, " in general it is desirable that there should be a distance of at least 25 feet between such trees and a building " was misleading. In fact, if a single figure was desired, the " safe distance " for buildings with shallow foundations on clay would be 100 feet.

Recent experience was indicating that the cost of a short bored foundation was no more than that of a traditional footing foundation and since the former had the advantage of guarding against the effects of tree shrinkage, provided the piles were deeper than the root system or groundwater level, it was to be hoped that the pile foundation would become standard practice in clays.

The Author was grateful to those who had contributed brief records of cases where trees had caused shrinkage cracking. The examples given by Mr King and Mr Creswell were particularly useful, and the following remarks concerning those cases could be made.* *Case I*.—The cracking in the back wall of the house, 74 feet from the trees, was slight. On the other hand, 1-inch diameter roots were found 64 feet from the trees under wall No. 1 (see *Fig. 6*), and it would seem reasonable to assume that the " safe distance " in that case was not less than 75 feet. *Case II*.—One wing of the building had cracked at a distance of 80 feet from the oaks, but it was not certain that that crack could be attributed to the trees. The large shrubs might have been responsible. But there was no doubt that the general damage to the building was caused by the oak trees, and

* Mr King had kindly supplied further information by private correspondence.

TABLE 3.—EXAMPLES OF SEVERE CRACKING OF BUILDINGS ON LONDON CLAY

Site	Building	Depth of footings	Trees		Approx. distance to major cracking (L): feet	$\frac{L}{H}$	Ground cover between trees and building
			Distribution and and species	Height (H): feet			
Northolt	Two-storey	3′ 8″	Row of black poplars	32	34	1·1	Grass
Stamford Hill	60-foot brick wall, theatre	4′ 0″	Row of Lombardy poplars	35	35	1·0	Tarmac
Hornsey	Three-storey house	4′ 0″	Row of Lombardy poplars	55	75	1·4	Paved
Harrow	Two-storey	3′ 0″	Row of oaks	50	50	1·0	Grass
Edgware	Single-storey	3′ 6″	Row of elms	55	65	1·2	Tarmac
Pinner	Single-storey	2′ 6″	Row of elms	50	70	1·4	Tarmac
East Finchley	Single-storey	3′ 0″	Single oak	40	40	1·0	Grass
Northwood	Two-storey house	2′ 6″	Row of black poplars	60	80	1·3	Grass

TABLE 4.—EXAMPLES OF ABSENCE OF CRACKING IN BUILDINGS ON LONDON CLAY

Site	Building	Depth of footings	Trees: Distribution and species	Trees: Approx. Height (H) : feet	Approx. distance from trees (L_0) : feet	$\frac{L_0}{H}$	Ground cover between trees and building
Northolt	Two-storey	3′ 8″	Row of black poplars	32	38	1·2	Grass
Stamford Hill	Three-storey school	3′ (approx.)	Row of Lombardy poplars	42	62	1·5	Tarmac
Edgware	Single-storey	3′ 6″	Row of elms	55	80	1·5	Tarmac
East Finchley	Single-storey	3′ 6″	Single Lombardy poplar	25	25	1·0	Grass
Stamford Hill	Three-storey school	9′ (approx.)	Row of Lombardy poplars	42	20	0·5	Tarmac
Edgware	Single-storey	10′	Row of elms	55	60	1·1	Tarmac

he damage extended up to 50 feet from them. *Case III.*—Severe cracking t the north end of the building occurred at a distance of 65 feet from the rees, but at the south end, situated about 80 feet from the trees, only hair racks could be found. Ground-water level was not far below the wall ootings, and the basement was therefore well below water level.

The above data, together with the information from Northolt and Stamord Hill, were expressed in Table 3, and it was clearly seen that, in spite of he footings all being between 3 feet and 4 feet deep, severe cracking ccurred at distances of 1·0 to 1·4 times the height of the trees.

Since writing the Paper the Author had visited several sites with Mr Ward, and data concerning three buildings* had been included in Table 3. n Table 4 information had been collected together concerning cases where o cracking had been observed; and the data given in those Tables led he Author to the conclusions given earlier.

Finally, the Author wished to say that the clear recognition of the langers of clay shrinkage caused by trees, and a proper understanding of he methods of preventing such dangers, were matters of outstanding mportance to architects and engineers concerned with housing, schools, nd other small buildings, and the credit for discovering that phenomenon nd for developing the bored pile solution was due to the Building Research Station, whose lead was now being followed by other countries.

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* A photograph of the building at East Finchley is reproduced as *Fig. 10* in eference 23.

CORRESPONDENCE
on a Paper published in Proceedings, Part I, March 1954

Paper No. 5971

"The Construction of Two Concrete-Lined Tunnels for the Machkund Hydro-Electric Project, South India" †

by

George Alexander Gauld, B.Sc., M.I.C.E.

and

Roger John Newport, B.Sc., A.M.I.C.E.

Correspondence

Mr A. B. Harman observed with regard to the concreting of the arch and walls of the tunnels by means of the pneumatic placer, he had witnessed lining defects such as those described by the Authors in two tunnels similar to those of the Machkund Project. In the instances in question the concrete was mixed outside the tunnels in two 1-cubic-yard mixers and transported in transit mixers each holding nearly 3 cubic yards. With three pairs of transit mixers and three Diesel locomotives, the supply of concrete to the pneumatic placer enabled rates of placing exceeding 40 cubic yard per hour to be achieved. A mix having 3-inch to $3\frac{1}{4}$-inch slump wa found to be satisfactory and occasional mixes of 2-inch slump did no cause blockages. That was for a shutter length of up to 80 feet and he wa surprised to learn that the Authors had found it necessary to use a mi of 4-inch to 5-inch slump to avoid blockages with a shutter length of onl 40 feet. The adequacy of the air supply was probably important in tha respect, but the Authors might have been handicapped by having largel to use crushed stone sand.

The main objection to the pneumatic placer as used in the Machkun and other similar projects appeared to be that a triangular area of porou concrete was commonly found at the starting end of each pour, indicatin that segregation occurred until the concrete had built up to crown leve The Authors, in commenting on that, had said that little trouble wa experienced once the discharge pipe was buried. Whilst Mr Harma

† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 111 (March 1954).

agreed on the desirability to keep the pipe buried, he thought the emphasis should be on the fact that once the concrete had built up to crown level, the discharge was into freshly placed concrete, whether the end of the pipe was buried or not, instead of against the hard construction joint of the previous pour.

Good compaction in the tunnel arch appeared to be provided by the discharge of the placer, and in the walls by hand punning with, or without, vibration ; but, in Mr Harman's experience, honeycombed patches were liable to occur in the region of springing level and adequate access for some form of compaction in that area was most important. As the Authors had pointed out, delays during concreting appeared to have been a contributory factor of some importance. He thought they might have found the best solution in the form of closely supervised hand punning in conjunction with external vibration, the former being the more important, particularly with the rate of placing which they quoted, and the latter being mainly a means of improving the surface finish. It should be borne in mind that the mix workability suitable for the pneumatic placer was not normally associated with compaction by vibration and Mr Harman had found that when internal vibrators were used, their main function appeared to be to keep the concrete moving along the shutter ; that helped to prevent the formation of pour planes, and the occurrence of "tear shears" when concrete which had built up too steeply towards the point of discharge slumped along the shutter.

In spite of certain difficulties, Mr Harman thought there would be an increasing tendency in the future to use the pneumatic placer in favour of the concrete pump for the concrete lining of tunnels. Its main advantages were the much higher rate of placing which it achieved and its freedom from mechanical breakdown ; but other factors might be important in some circumstances, such as the fact that it was not nearly so sensitive as the pump to mix workability and aggregate grading, and appreciably less head-room was required for its operation.

The Authors thanked Mr Harman for his contribution but did not feel called upon to make a reply.

OBITUARY

FRANCIS MAURICE GUSTAVUS DU-PLAT-TAYLOR who died on the 22nd May, 1954, was born on the 7th December, 1878. He was educated at Winchester College, and at London University.

In 1904 he was appointed Assistant Engineer and Acting Resident Engineer to the Mersey Docks and Harbour Board on new works costin £1,000,000.

Between 1904 and 1909 he was Resident Engineer for the East an West India Docks of the London and India Docks Co. In 1909 he becam Resident Engineer at Tilbury Docks for the Port of London Authority From 1924 he was in private practice and carried out sea-defence works i Kent and Sussex, river training and drainage schemes, and land reclamatio projects. He also carried out numerous arbitrations involving large sums and acted as expert witness and as Court Expert.

During the 1914–18 war he served in the Royal Artillery and becam Instructor in Gunnery and Commandant of the siege artillery reinforcemen training school in France.

Mr Du-Plat-Taylor was elected a Member in 1919 and presented thre Papers [1, 2, 3], to the Institution. He was awarded a Telford Premiu for his Paper on "The Prevention of Coast Erosion," [1] and a Man Premium for his Paper on "Extensions at Tilbury Docks, 1912–1917." He was also a Member of the Institution of Mechanical Engineers, a Fello of the Institute of Arbitrators, a Past President of the British Section o the Société des Ingenieurs Civils de France, a Member of the Association o Consulting Engineers, and a Fellow of the Royal Society of Arts.

In addition to two handbooks for Artillery regiments, Mr Du-Plat Taylor wrote two books, "The Design, Construction and Maintenance o Docks, Wharves and Piers" and "Reclamation of Land from the Sea." He presented Papers to the Société des Ingenieurs Civils de France and t the Institution of Structural Engineers.

From 1937 up to the time of his death, Mr Du-Plat-Taylor was member of the Scientific Development Committee of the Royal Nation Institute for the Blind, and from 1945 to 1949 was Chairman of that Com mittee. The development of apparatus for the Blind was in no sma measure due to his engineering knowledge and intiative.

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[1] J. Instn Civ. Engrs, vol. 15 (Nov. 1940), p. 53.

[2] Min. Proc. Instn Civ. Engrs, vol. 215, p. 165 (1922–23).

[3] "The River Rother Improvement Works," Sel. Engng. Paper No. 197, Inst Civ. Engrs, 1930.

In 1938 he took the Oaths as Magistrate in the county of Surrey and served in that capacity until December 1953, when his name was transferred to the Supplemental List. He was Chairman of the Mortlake Bench from 1950 to 1953.

He is survived by his wife, one son and one daughter.

JOHN LEONARD EVE, who died at Wimbledon on the 25th June, 1954, was born at Aveley in Essex, on the 3rd February, 1887. He was educated at University School, Hastings, and at the Technical Institute, Folkestone.

From 1904 to 1906 he served an apprenticeship at the Folkestone Electricity Supply Co. Ltd, at the end of which he was appointed Engineer in Charge of Shift at the generating station. He later became Superintendent at High Wycombe power station. From 1910 he was Assistant Engineer with Robert W. Blackwell & Co., on the London, Brighton, and South Coast Railway electrification. In 1914 he became Chief Constructional Engineer and in charge of lay-out design for British Insulated Cables Ltd, on Melbourne Railway electrification. In 1924 he was appointed Departmental Chief Engineer to The Foundation Co., London, and was in charge of design and construction of all the Scottish river crossings for the C.E.B. grid scheme. In 1930 he formed his own company.

Mr Eve was elected a member in 1939. For his Paper, in collaboration with Mr R. C. Brown, on " The Erection of Tall Towers ",[1] he was awarded a Telford Premium. He was also a member of the Institution of Electrical Engineers.

In 1915 he married Anne Gill, daughter of John Blackmore Gill, by whom he had a son and a daughter. His first wife died in 1937 and in 1940 he married Doris Matthews, daughter of Charles W. Matthews who survives him with a son.

Sir ARTHUR WATSON, C.B.E., who died in Exeter, Devon, on the 13th April, 1954, was born at Manchester on the 18th September, 1873. He was educated at Manchester Grammar School, and at Victoria University, Manchester.

In 1893 he was appointed Assistant Resident Engineer with the Lancashire and Yorkshire Railway Company, and as Resident Engineer from 1900–1905 was in charge of extensive railway construction works. In 1910 he was promoted to be Superintendent of the line, and General Manager in 1919; 4 years later he became the first General Manager of the London, Midland and Scottish Railway.

During the first World War he served with the rank of Lieutenant-Colonel in the Corps of Royal Engineers and acted as liaison officer between the railways and the Quarter-Master General.

[1] J. L. Eve and R. C. Brown, "The Erection of Tall Towers," Struct. and Bldg Paper No. 11, Instn Civ. Engrs, 1944–45.

Sir Arthur was elected an Associate Member in 1899, and was transferred to the class of Member in 1911. For his Student's Paper on "Maintenance of Railway-Tunnels,"[1] he was awarded a Miller Prize. He was also a member of the Advisory Council of the Board of Trade, a member of the Permanent Commission of the International Railway Association, and a founder member of the Institute of Transport.

He is survived by two daughters.

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[1] Min. Proc. Instn Civ. Engrs, vol. 110, p. 180 (1895–1896).

CORRIGENDA

Proceedings Part I, January 1954.

Paper No. 5939 (Worthington), p. 48, equation (5)

For $$\left(\gamma - \frac{P}{r^2}\right)\frac{d\theta}{dx} - \alpha\frac{d^3\theta}{dx^3} = 0$$

read $$T = (\gamma - Pr^2)\frac{d\theta}{dx} - \alpha\,\frac{d^3\theta}{dx^3}$$

Proceedings, Part 1, May 1954.

p. 342, line 12 from bottom, *for* "square inch" *read* "square foot."

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